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Literature Review on Seismic Performance of Building Cladding Systems

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ABSTRACT

A literature survey on the seismic performance of building cladding systems was conducted. The focus was on heavy cladding panels, with a particular emphasis on precast concrete cladding panels.

The references used in this literature survey were identified by using the following resources: (1) computerized library data bases, including the "melvyl" system for the University of California libraries, "eea" (earthquake engineering abstracts) available through "melvyl," and the "gladis" system for the U.C. Berkeley libraries; and (2) the CD-rom from the Information Service at the National Earthquake Engineering Center (NCEER) at SUNY at Buffalo that contains abstracts for references shelved there and at the EERC Library.

The facilities used to retrieve the references of interest included: (1) the U.C. Berkeley libraries, including the Earthquake Engineering Research Center (EERC) Library at the Richmond Field Station, the Engineering Library on the U.C. Berkeley campus; and the Environmental Design Library on the U.C. Berkeley campus; (2) the Information Service at NCEER; (3) the National Technical Information Service (NTIS) at the U.S. Department of Commerce; and (4) the Prestressed/Precast Concrete Institute (PCI) in Chicago, Illinois.

At the Environmental Design Library, the following additional resources were found to be helpful: the Avery computer data base for post-1978 references, the Avery printed books for pre-1978 references, the Art Index on CD-rom, and the Construction index book series.

Some of the key words used in the search included: precast, cladding, reinforced concrete, concrete, facades, skins, siding, etc.

The literature survey is organized as follows: Chapter 1 is an introduction that includes definitions, cladding panel configurations, details of architectural precast concrete cladding systems in the U.S.A., New Zealand, Japan, and Canada. Chapter 2 describes the current practice for seismically isolated precast concrete cladding panels and connections, including U.S. codes and their interpretation and foreign codes. Chapter 3 offers information on the structural utilization of precast concrete cladding panels and connections, including an historical overview, levels of contribution in seismic response, architectural implications for structural cladding, conditions for effective structural cladding, and issues of responsibility. Chapter 4 contains abstracts and informational highlights from research on the structural utilization of precast concrete cladding panels and connections, including eleven sets of research projects from the U.S.A., one project from Canada, and one project from Japan. Chapter 5 outlines other cladding materials for heavy panels, including prefabricated panel systems, GFRC panels, new types of reinforcement, a new type of RC sandwich panels, and steel and steel alloy panels.

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CHAPTER 1

USE OF ARCHITECTURAL PRECAST CONCRETE CLADDING PANELS IN HIGH SEISMIC ZONES

1.1 Introduction

"Facade design uses an architectural language that has existed for thousands of years: the pattern of voids (windows) and solids (opaque materials). This pattern has to be related to the functional need of planning and interior spaces, for the glazing project light and, until the recent advent of air conditioning, ventilation. When an architect thinks about facade design, he immediately begins to conceive patterns, that are both geometrical, materials related and colorful," as noted by Arnold [1989].

Arnold continued, "There are three basic typologies for determining the pattern of glazing and opaque materials in a buildings, based on the predominant expression or emphasis of the facade geometry. The first typology is that of vertical emphasis; the second is horizontal expression; and the third is rectangular, in which the facade is either a rectangular grid, or involves a pattern of rectangular windows inserted into a plane surface. Sometimes one of these patterns forms the entire facade; in other cases the facade is composed of two or three of these typologies in combination."

Levy [1989] discussed past and current examples of precast facade panels. He offered historical information on grid walls dating back to the 1960s, tube buildings, shear field panels, composite panels, and grid walls revisited. He included several figures showing structural details.

As noted by Sack, *et al.* [1989], "In the mid-1970s there were basically two types of architectural precast wall panels: the window box, which is a one-piece component enclosing on bay of the structural frame work; and the articulated panel used to clad individual spandrels and columns. The Trans-American and Hartford Buildings of San Francisco exemplify the window box panel system and the Bank of Tokyo and 595 Market Street Building also of San Francisco, illustrate the use of articulated panels. A subsequent trend used only spandrel beam panels and incorporated continuous glass about the building at each floor; the approach eliminated the need to design the panels to withstand stressed in the connections associated with interstory drift. Today, we see a wide range of architectural precast concrete cladding."

1.1.1 Examples of the Use of Architectural Precast Concrete Cladding

Freedman [1990] stated that typical wall panel system cross sections can be conventional walls, sandwich panels, and rain screen walls. From exterior to interior, conventional walls are comprised of a panel with its finish, studs and insulation, a membrane, and gypsum board;

sandwich panels are comprised of an exterior concrete wythe, insulation, structural interior concrete wythe, (an optional) metal furring strip, and (optional) gypsum board; and rain screen walls are comprised of a vented exterior facing of stone, clay product, or precast, an air gap cavity, insulation, structural interior concrete wythe, (an optional) metal furring strip, and (optional) gypsum board.

These wall panel systems may be solid wall panels, window wall panels, or spandrels. In addition, column covers and mullions are a common application of cladding units. As also noted by Arnold [1989], Freedman stated, "In high-rise buildings, three characteristic facade patterns can be identified that impact considerably on panel design. The first is that of cladding that plates the structural framing, vertically and horizontally, the large opening then being in-filled with glass.

"The second pattern eliminates the column covers, and the facade then becomes alternating horizontal band of spandrel panels and glazing. In this pattern, the panels and glazing are placed in front of the column which are thereby suppressed.

"The third pattern is a return to the traditional facade design of rectangular window openings 'punched' into a plane surface. This pattern originated from the requirement of loadbearing walls - that wall area must be provided between glazing to carry vertical loads - so windows were relatively small. The reappearance of this pattern derives some rationale from the needs of energy conservation which mitigates against large areas of poorly insulated glazing. A much stronger impetus comes from the dictates of architectural fashion and the desire to return to modelled faces and the visual interest that can be obtained by the traditional manipulation of voids and solids. This trend has results in some ingenious precast concrete configuration with the use of L- and T- shaped panels to reduce the number of costly joints. These panel shapes are derived from the requirements of erectors and their efforts to reduce installation cost.

To find examples of precast concrete or other "heavy" cladding panels in U.S. seismic zones 4 and 3, and comparable zones abroad, an extensive library data base search was done to identify magazine and journal articles. Desirable articles would have included exterior panel elevation photographs and/or drawings, horizontal and vertical sections through the panels and perimeter structural framing showing cladding connection details, plans and sections of structural framing including the foundation, etc. Unfortunately, published articles were found not to contain this type of comprehensive information. Architecture magazines contain exterior and interior photographs, and drawings of architectural plans and sections, but rarely contain information on the structural framing and building cladding. Civil and structural engineering magazines tend not to include articles on cladding, unless there has been a dramatic, expensive failure, or there have been numerous failures or signs of distress of the same type. Engineering magazines published by the concrete industry only contain short articles on precast concrete cladding panels and connections, only if a technological improvement or innovation has been introduced, or if precast concrete cladding panels have been used in an usual or atypical manner. Included in this section are citations to the few articles that were identified.

Wallace [1987] authored an article on a redesign using smaller precast concrete panels that improved constructability and enabled steel erection to proceed earlier. These smaller precast concrete panels were designed and detailed to participate in the lateral load resistance with the structural framing.

"Erecting 3-story-high tilt-up panels that weigh about 100 tons each requires an extremely large crane and many large braces once the panels are lifted. Smaller panels would be easier and cheaper to handle, but how do you make them smaller when large window openings penetrate almost the entire width of each panel? This was the question concrete contractor A.T. Curd Builders, Inc., asked when reviewing the designs for the Hughes Aircraft Sunny Hills Expansion in Fullerton, California... The expansion consisted of two connecting office buildings, each designed as an interior steel frame with exterior concrete tilt-up panels. The tilt-up panels were both structural and architectural. Vertical loads were carried by the interior steel frame, but all lateral loads were carried by the concrete facades. To provide the resistance to lateral loads required by a building located in Seismic Zone 4, the exterior concrete panels had to be welded to the foundation and seam-welded at the vertical joints.

"The tilt-up panels for these three-story buildings were design about 30 feet wide and 60 feet high. Casting and erecting such large panels would not only be costly, but the required bracing would interfere with steel erection. A.T. Curd Builders, Inc., proposed a more constructable method. With the cooperation of the structural engineer, they redesigned the concrete exterior. In essence, they cut the large tilt-up panels into four smaller precast components: 3-story high column panels, first-floor retaining wall panels, spandrel panels and parapet panels. The panels still had to be welded together and connected to the foundation, but instead of weighing 100 tons per piece, they ranged from 12-ton parapet panels to 45-ton column panels...

"To resist shear loads between panels, the column panels were designed with heavily reinforced haunches and the walls panels with blockouts that fit around these haunches. Moment resistance was provided by continuous horizontal reinforcing bars welded at the joints between columns and panels...

"The precast exterior was connected to the interior steel frame by steel plates embedded in the column panels. High-strength bolts protruded through the plates for connection to the structural steel. These bolts carried the vertical shear loads."

Further information can be found in Wallace [1987].

Harriman [1991] wrote an article on "architects who are designing precast concrete forms that promote innovative applications of the material." One example each from Washington, D.C., Boston, and San Francisco are given.

In San Francisco, Heller & Leake Architects were responsible for the cladding on 55 Stockton Street. "Turning a corner of Union Square in downtown San Francisco, 55 Stockton is designed to be contextual, responding to the ornate terra-cotta facades of its neighbors. The new mixed-use building related to the character and scale of it surroundings with a highly articulated facade composed of sandblasted white precast panels that recall the forms of the late 19th-century cast-iron commercial structures."

As noted by Harriman, "'The prominence of a grid of joint lines on a previous project taught us a lot about precast,' explains project architect Michael Garcia. 'With flat or simple facades, the grid can enhance the architecture, but can be a detriment to esthetic intentions of a highly ornamented facade if panel sizes are not carefully considered.' To prevent joints from dominating the facade, the architects designed precast element to intersect along column edges, floor lines, and window mullions. In addition to accommodating cladding expansion and contraction, the typical $^{3}/_{4}$-inch joint spacing was chosen to comply with seismic codes. The cladding is attached to a structural frame with push-pull connections, in which a rod is threaded through an enlarged hole in a clip, allowing vertical adjustment."

The figures of a wall section, corner detail, and column detail are conceptual. That is, they show the location and relative size of the precast concrete panels in relation to the perimeter steel beams or columns, firestopping, batt insulation, etc. The figures do not include the cladding-to-frame connections.

Knowles [1990] described "the design, detailing, fabrication and installation of glass fiber reinforced concrete (GFRC) architectural panels for a 42-story hotel, (the San Francisco Marriott Hotel, which is) the largest GFRC clad building of its kind in the United States... A major feature of this project, and the subject of this article, is the use of 340,000 sq. ft. (31,620 m^2) of GFRC architectural cladding panels. Altogether, 2,400 GFRC panels were required... The GFRC panels weighed about 20 psf (98 kg/m^2), which is approximately one-quarter the weight of regular architectural precast concrete panels. As a result, the lightweight panels reduced the structural steel requirements by 100 to 150 tons (71 to 136 t)... The types of GFRC units consist of window wall panels, solid wall panels, spandrel panels, and column covers. The window wall panels and solid wall panels are approximately 10 ft. in height by 18 ft. in length (3.05 m x 5.49 m). The spandrel panels are about 5 ft. in height by 18 ft. in length (1.52m x 5.49 m) and the column covers are approximately 10 ft. in height by 3 ft. in width (3.05 m x 0.91 m). The panels varied in thickness from 8 to 24 in. (203 to 610 mm)... The panels are comprised of a GFRC skin, with an architectural face mix, attached to a 6 in. (152 mm) steel stud frame... The steel stud frame was fabricated using structural steel tube members and galvanized light gauge steel studs. The steel stud frame stiffens the GFRC skin and provides a surface for the attachment of the interior finishes. The steel stud frame also provides support for the attachment of the glazing system and the louvers."

Knowles continued, "Bearing connections, either angle or structural tube, were welded to the structural tube members of the steel stud frames. These connections were attached to steel floor beams in pockets in the concrete floor slab. Lateral (or push/pull) connections were all-thread rods, threaded into nuts welded to the structural tube members. These connections were bolted to angles, structural tubes or channels, welded to either the bottom of the floor beams of the steel columns."

The figures in the paper may be reductions of larger drawings. They are not easily readable from photocopies and are not included herein. The interested reader is referred to Knowles [1990] for the figures on GFRC skin drawings for window wall panel, steel stud frame rear view and sections for window wall panel, window wall panel section, and spandrel panel section.

Rihal [1988a] included photographs of building exteriors in his report. The photographs include seven medium-rise (up to 10 story) buildings in Los Angeles, California, and a medium-rise building in San Jose, California. The configurations of cladding include: vertical patterns with column covers emphasized, full-bay full-story window wall patterns with up to five windows per bay, a checker-board pattern, a horizontal pattern with spandrel covers emphasized, and a configurations with both column and spandrel beam panel covers. The interested reader is referred to Rihal's report, because the photocopies of the photographs could not be adequately reproduced for inclusion here.

PCI [1989] includes color photographs of exterior facades, most of which are not identified by location. These color photographs do not photocopy well, and can be seen in PCI [1989].

1.2 Definitions

Information is offered by PCI [1988, 1989, 1992] on architectural precast concrete cladding panels, as well as by Freedman [1990]. The primary source for information is PCI [1989]. Definitions of interest to this literature survey include:

Architectural precast concrete refers to any precast concrete unit of special or occasionally standard shape that through application or finish, shape, color or texture contributes to the architectural form and finished effect of the structure; units may be structural an/or decorative, and may be conventionally reinforced or prestressed.

Bearing (direct and eccentric) connections are intended to transfer vertical loads to the supporting structure or foundation. Direct bearing connections are used primarily for panels resting on foundations or rigid supports where movements are negligible. Eccentric bearing connections are usually used for panels above the first support level when movement of the support system are possible.

Cladding (non-loadbearing panel) is a wall unit that resists only wind or seismic loads and its own weight (but not the gravity loads from the structural framing).

Connections are a structural assembly or component that transfers forces from on precast concrete member to another, or from one precast concrete member to another type of structural member.

Non-loadbearing is a term used to indicate that precast concrete cladding panels do not support gravity loads from the building framing. The term can be used with *architectural* or *structural* precast concrete cladding panels.

Sandwich cladding panel is similar to a *sandwich wall panel,* which is a wall panel consisting of two layers (wythes) of concrete fully or partly separated by insulation. An example is given by Eina, *et al.* [1994].

Structural precast concrete cladding panels are used as part of the lateral load-resisting framing. The panels are used as shear panels with connections intentionally designed to resist a prescribed level of story shear.

Tie-back (lateral) connections are intended to keep the precast concrete panel in a plumb or other desired position and resist wind and seismic loads perpendicular to the panel.

Figure 1.1 (taken from fig. 1.4.1, PCI [1989]) gives the terminology for precast concrete units, for both typical and sculptured panels as cast in the face-down position.

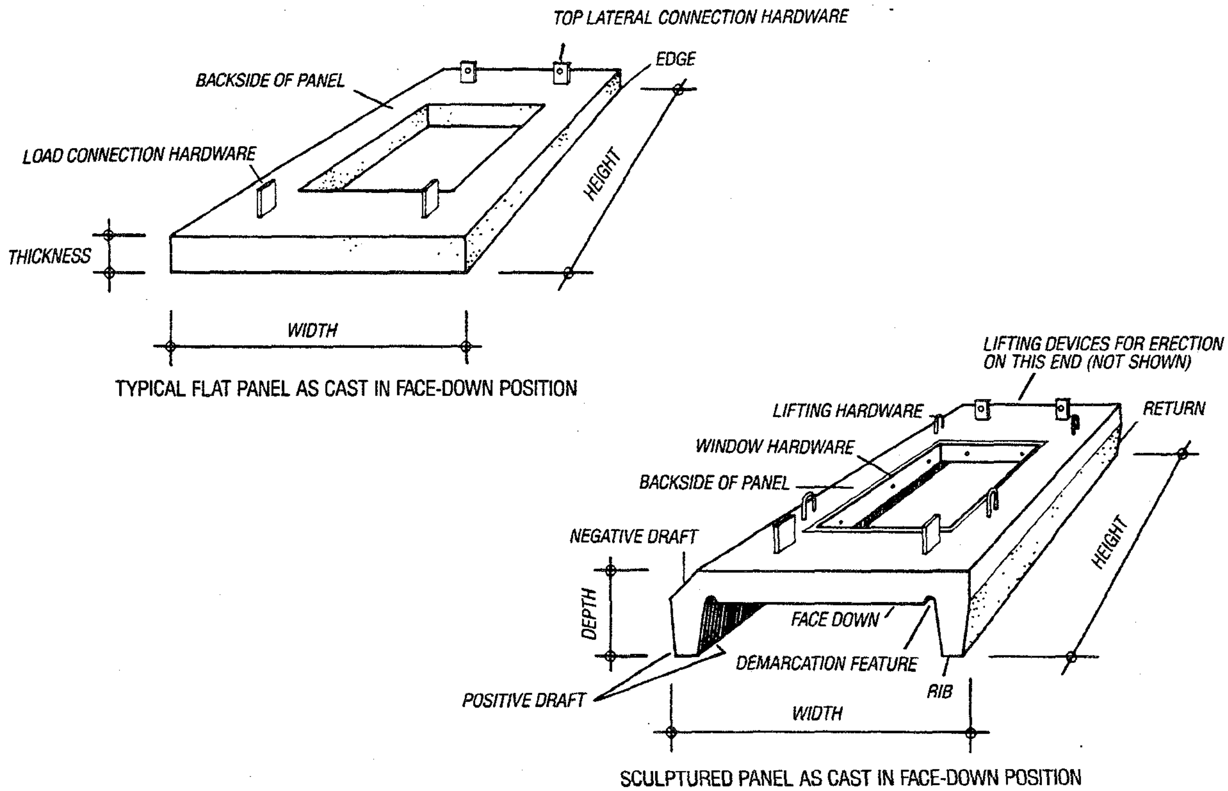


Figure 1.1. Terminology for precast concrete units (from PCI [1989]).

1.3 Cladding Panel Configuration

According to PCI [1989], "The use of non-loadbearing precast concrete cladding... has been the most common application of architectural precast concrete. Cladding panels are those precast elements which resist and transfer negligible load from other elements of the structure. Generally, they are normally used only to enclose space, and are designed to resist wind, seismic forces generated from their self weight, and forces required to transfer the weight of the panel to the support. Cladding units include wall panels, window wall units, spandrels, mullions and column covers. Their largest dimension may be vertical or horizontal. These units may be removed from the wall individually without affecting the stability of other units of the structure itself. For the purpose of the discussion, cladding or curtain wall units do not extend in height beyond a typical floor-to-floor dimension and are normally limited in width to less than the bay width of the structure.

"Typical wall panel system cross section sections are shown in figure 1.2 (taken from fig. 2.5.1, PCI [1989]). These walls may be solid wall panels, window wall panels or spandrels. In addition, column covers and mullions are a common application of cladding units.

"In high-rise building three characteristic facade patterns can be identified that impact considerably on the panel design. The first is that of cladding that plates the structural framing, vertically and horizontally, the large opening then being infilled with glass (see fig. 1.3 taken from fig. 2.5.2, PCI [1989]).

"The second pattern eliminates the column covers, and the facade then becomes alternating horizontal bands of spandrel panels and glazing (see fig. 1.4 taken from fig. 2.5.3, PCI [1989]). In this pattern the panels and glazing are placed in front of the columns, which are then individually suppressed.

"The third pattern is a return to the traditional facade design of rectangular window openings 'punched' into a plane surface (see fig. 1.5 taken from fig. 2.5.4, PCI [1989]). This pattern originated from the requirement of loadbearing walls, that wall area must be provided between glazing to carry vertical loads, and so windows were relatively small. The re-appearance of this pattern derives some rationale from the needs of energy conservation which mitigates against large areas of poorly insulated glazing. A much stronger impetus comes from the dictates of architectural fashion and the desire to return to modeled facades and the visual interest that can be obtained by the traditional manipulation of voids and solids. This trend has resulted in some ingenious precast concrete configurations with the use of L- and T- shaped panels to reduce the number of costly joints. These panels shapes are derived from the requirements of erectors and their efforts to reduce installation cost. Some typical panel arrangements are shown in figure 1.6 (taken from fig. 2.5.5, PCI [1989])."

Figure 1.2. Typical wall systems (from PCI [1989]).

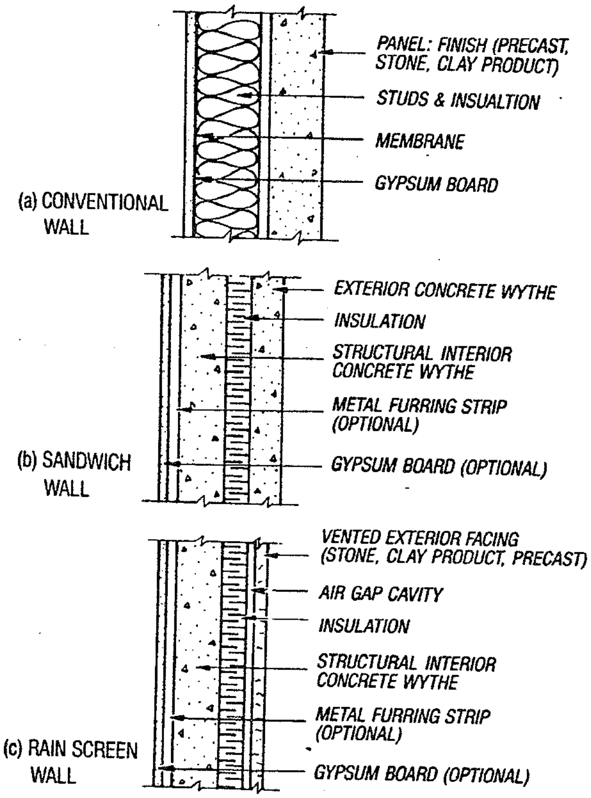


Figure 1.3. A characteristic facade pattern: spandrel panels and column covers (from PCI [1989]).

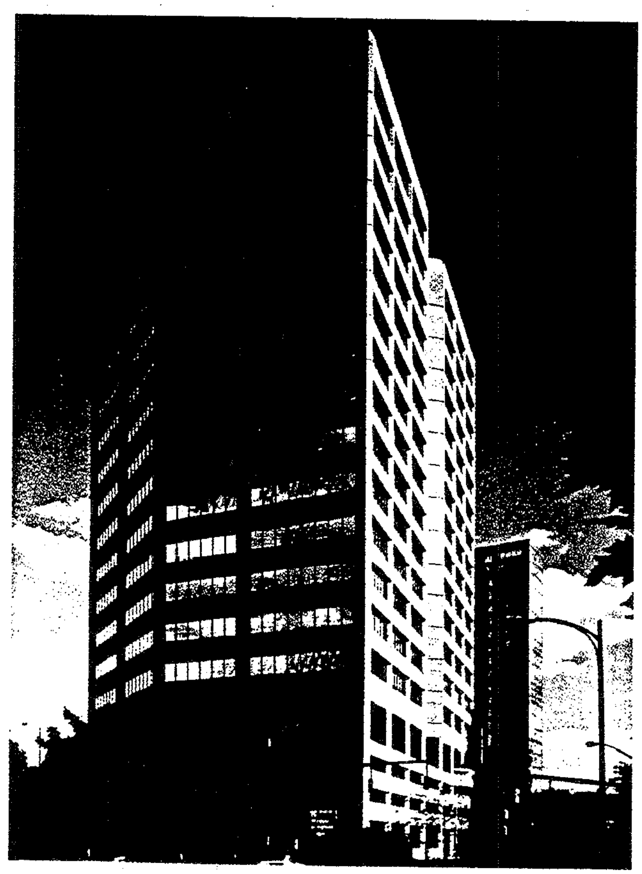


Figure 1.4. A characteristic facade pattern: spandrel panels and glazing (from PCI [1989]).

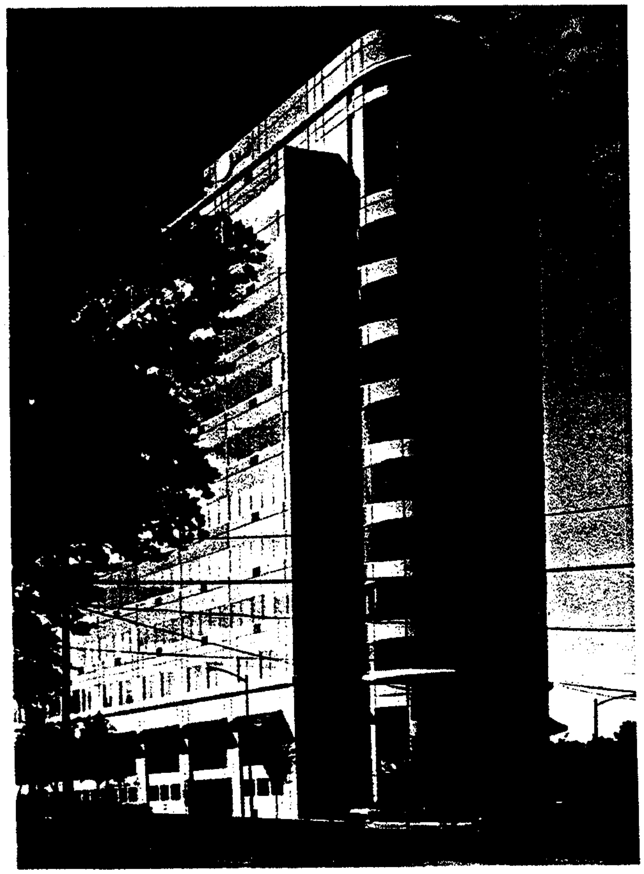


Figure 1.5. A characteristic facade pattern: traditional design (from PCI [1989]).

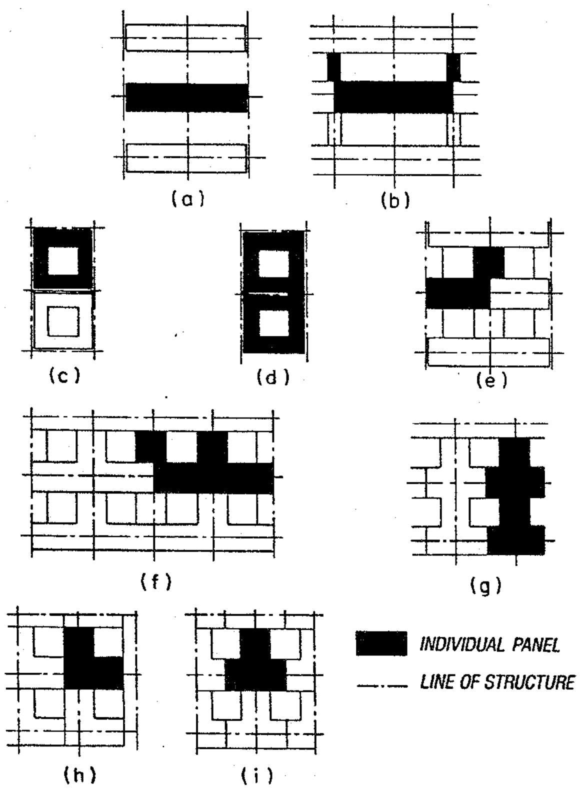


Figure 1.6. Typical arrangements of precast concrete panels (from PCI [1989]).

1.4 Details of Architectural Precast Concrete Cladding System

As noted in PCI [1989], "The cost of hardware is mainly governed by load requirements including special structural functions and possible earthquake conditions. Hardware cost may be minimized by making the precast concrete units as large as is consistent with the size limitations (see cited reference, sec. 3.3.9, 4.2.9). Four connections are the minimum required for most precast concrete units. The labor cost of producing and handling small individual pieces of hardware normally exceeds the material costs making the relative cost of hardware high for small units."

1.4.1 Cladding Panels and Connections: U.S.A.

Before giving specific information from fabricators and engineers on cladding panels and cladding connections, a case study carried out twenty years ago by an architectural firm is briefly introduced, primarily to offer the reader a different perspective, starting from the conceptual design phase of building design. In the 1990s, the design phase is most often carried out solely by architects. There are many reasons for the absence of structural engineers, but these, and implications on design development, etc., are outside the scope of this literature survey.

McCue, *et al.* [1978] prepared a report on the architectural design of building components for earthquakes. In the case study on building response and component design for an enclosure wall, the authors stated that this material "illustrates use of the Dynamic Model (a four-part model which describes the various elements of a building, their interactive relationships during earthquakes, and the effect of the interaction on overall building response) when it was in its preliminary stages of development. The Model was used as an aid in the design of an enclosure wall for an actual building being designed concurrently with research done under the study team's first NSF grant. Included in the case study are descriptions of the seismic conditions imposed by the site, design of the basic structural system, detailed component design, mock-up testing, and fabrication and construction of the enclosure wall at the actual site. All of these activities are described in terms of the effect of the Dynamic Model on the design process."

1.4.1.1 Cladding Panels: U.S.A.

According to PCI [1989], "Non-loadbearing panels are those precast concrete units which transfer negligible load from other units of the structure. Generally they are closure panels only, and are designed to resist wind, seismic forces generated from the self weight and forces required to transfer the weight of the panel to the support. It is rare that these externally applied loads will produce the maximum stresses; the forces imposed during manufacturing and erection will usually govern the design, except for the connections.

"All non-loadbearing panels should be designed to accommodate movement freely, and, whenever possible, with no redundant supports, except where necessary to restrain bowing.

"The relationship of the deformations of the panel and the supporting structure must be evaluated, and care taken to prevent unintended restraints from imposing additional loads. Such deformation of the supporting structure may be caused by the weight of the panel, volume changes in concrete frames, or rotation of supporting beams. To avoid imposing loads on the panel, the connections must be designed and installed to permit such deformations to freely occur.

More detailed descriptions of design considerations for deflection, bowing, wind loads, and frame shortening are given in PCI [1989].

According to PCI [1989], "In designing architectural precast concrete panels, it is desirable that there not be any discernible cracking... In members in which concrete stresses during service are less than the allowable flexural tension, distributed reinforcement is needed to control cracking that may unintentionally occur during fabrication, handling or erection and also to provide ductility in the event of an unexpected over-loading. In members in which the stresses are expected to be greater than the allowable flexural tension, conventional or prestressed reinforcement is required for satisfactory service load performance, adequate safety and meeting esthetic requirements. Reinforcement may serve either or both of these purposes in architectural precast concrete.

"The types of reinforcement used in architectural precast concrete wall panels includes welded fire fabric, bar mats, deformed steel bars, prestressing tendons and post-tensioning tendons. Non-prestressed reinforcement is normally tied or tack welded together into cages by the precast concrete manufacturer, using a template or jig when appropriate, unless the precast concrete unit is a simple flat panel. The cage, whether made for the entire casting or consisting of several sub-assemblies, must have sufficient three dimensional stability so that is can be lifted from the jig and placed into the mold without permanent distortion. Also, the reinforcing cages must be sufficiently rigid to prevent dislocation during consolidation in order to maintain the required cover over the reinforcement. The rigidity will normally improve with the tack welding and hence weldable grades of reinforcing steel are recommended. However, a designer should work with the grade of steel which is reasonably available to the precaster likely to bid on the project." More information is given in PCI [1989] on welded wire fabric, reinforcing bars, prestressing steel, shadow lines (steel reflection), tack welding, and corrosion protection.

Tawresey [1989] presented a general overview of structural considerations for curtain wall systems, including precast concrete panels and connections. Much of what he presented is offered in more detail by the references cited in Section 1.4.1.2 (below).

1.4.1.2 Cladding Connections: U.S.A.

According to PCI [1989], "The primary purposes of a connection are to transfer load to the supporting structure and provide stability. Precast concrete connections must also meet design and performance criteria. However, all connections are not required to meet precisely the same criteria. These criteria include:

1. *Strength:* A connection must have the strength to safely transfer the forces to which it will be subjected during its lifetime. In addition to gravity loads, the forces to be considered include:
   a. Wind and seismic forces.
   b. Forces from restraint of volume change strains.
   c. Forces induced into wall panels by restrained differential movements between the panel and the structure.
   d. Forces required for stability and equilibrium.
2. *Ductility:* This is the ability to accommodate relatively large deformations without failure. In connections, ductility is achieved be designing so that steel devices yield prior to concrete failure.
3 *Volume change accommodation:* Restraint of creep, shrinkage and temperature change strains can cause severe stresses on precast concrete members and their supports. These stresses must be considered in the design, but it is usually far better if the connection allows some movement to take place, thus relieving the stresses.
4. *Durability:* When exposed to weather, or used in a corrosive atmosphere, steel elements should be adequately covered by concrete, painted, galvanized, or epoxy coated. Stainless steel is sometimes used, but with a substantial increase in cost. All exposed connections should be periodically inspected and maintained.
5 *Fire resistance:* Connections, which could jeopardize the structure's stability, if weakened by a fire, should be protected to the same degree as that required for members that they connect.
6 *Constructability:* The following items should be kept in mind when designing connections:
   a. Standardize connection types.

b. Avoid reinforcement and hardware congestion.
c. Avoid penetration of forms, where possible.
d. Reduce post-stripping work.
e. Beware of material sizes and limitations.
f. Consider clearances and tolerances.
g. Avoid non-standard production and erection tolerances.
h. Standardize hardware items and use as few sizes as possible.
i. Use repetitious details.
j. Plan for the shortest possible hoist or crane hook-up time.
k. Provide for field adjustment.
l. Provide accessibility.
m. Use connections that are not susceptible to damage in handling.

"(For) an architectural precast concrete unit... used in a non-loadbearing function, various forces must be considered in design. For example, a cladding panel must resist its own self-weight, earthquake forces, when required, forces due to restraint of volume change or support system movement, and forces due to wind, snow and construction loads. If the panel is load-bearing, it must also resist and transfer the dead and live loads imposed on it by the supported structural members. These forces are transferred to the supporting structure through the architectural precast concrete panel's connections.

"Bearing pads are sometimes used to distribute loads over the bearing area and to accommodate construction, fabrication and erection irregularities. These pads reduce the concentration of forces at the connection by deforming readily within their thickness or allowing slippage. The physical characteristics of bearing pad material necessary to satisfy this function are: (1) Permanence and stability; (2) Ability to equalize uneven surfaces and avoid point pressure; and (3) Ability to accommodate movements.

"The pad supplier or precast concrete manufacturer should be consulted when selecting bearing pads. The type and material required will depend on the imposed loads and the expected relative movements of the cladding and support structure. The two most satisfactory materials are: (1) Elastomerics with known compression, shear, and friction strength and known ability to deform with movements; and (2) Plastics with low friction coefficients along with high compression and shear strength.

"If significant movements are expected, soft pads or low friction rigid pads should be used. However, if relative movement is not expected, a bed of rigid material such as grout or drypack can be used to make a bearing connection.

"*A designer should always remember that statically determinant design concepts are preferred. Simple connections will usually perform best.* One of the advantages of working with precast concrete is that connections may be design for specific purposes, and when properly designed, can be expected to perform accordingly.

"The principles for the design of connections are relatively easy to follow where precast concrete units are supported on one level, at two points, hereafter referred to as *Load Support or Bearing Connections,* and held in with some degree of flexibility at other points, hereafter referred to as *Tie-Back or Lateral Connections.*

"A common solution for floor to floor panels is to install load support connections near the bottom of the panel and place tie-back (lateral) connections at the top. Some designers prefer to have load support connections at the top and the lateral connections at the bottom.[1] This is common for spandrels. Lateral support at an intermediate level for tall, thin panels such as column covers is possible. In all cases, the basic connection concepts are similar.

"It is best to support the entire weight of the panel at one level. This is due to possible deflections of the supporting element. If supported by more than one floor, the varying deflections of supporting building frame members may cause the weight distribution to be indeterminate. Figure 1.7 (taken from fig. 4.5.1, PCI [1989]) illustrates ten basic design principles.

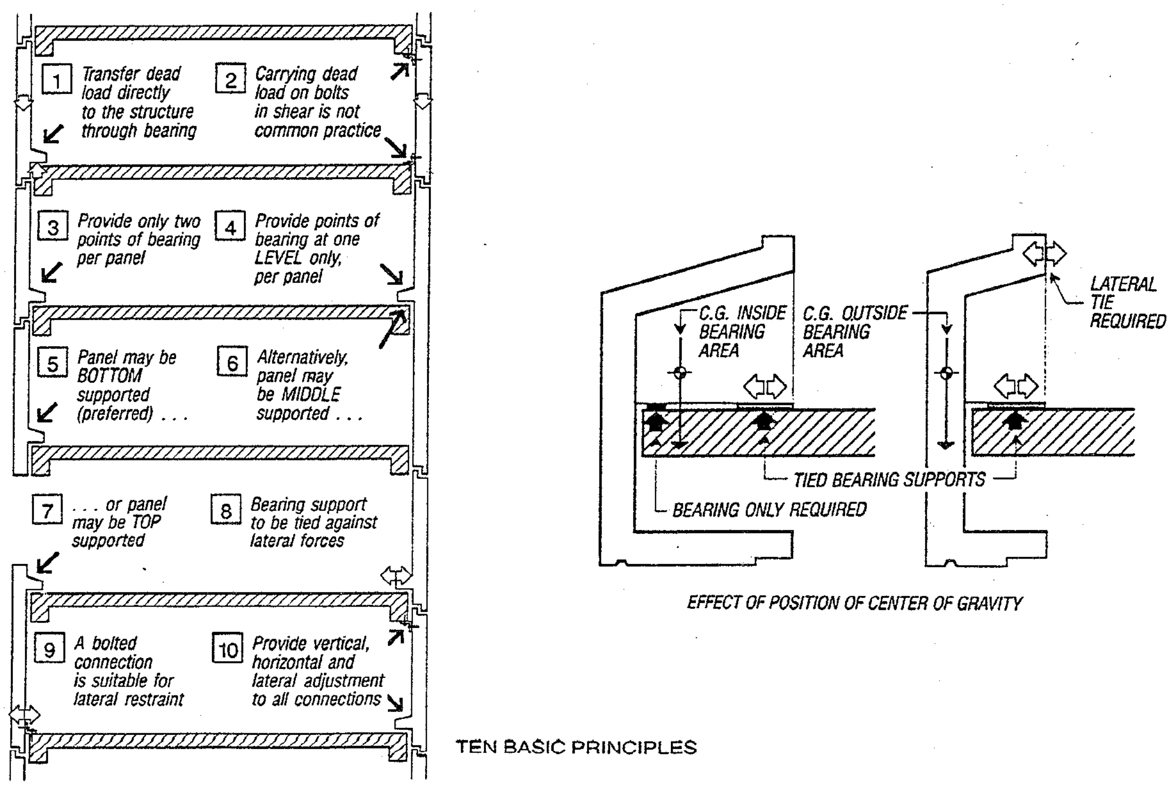


Figure 1.7. Design principles for cladding panel connections (from PCI [1989]).

1 Note: This is preferred by those structural engineers who are concerned about the loadbearing connections being installed at the bottom panel corners and the potential for the panels to rotate outwards about the bottom panel edge, either during construction when the top panels are not as yet installed, or during building service if the top panel connections were to fail.

"The arrangement and size of cladding elements with reference to the grid of the support system can vary. Since the panel size and the number and spacing of the connection points all influence the design, an optimum solution is desirable. In general, the largest possible size of panel with a minimum number of connections is the most economical, subject to limits imposed by handling, shipping, crane capacity and loads on the support system.

"Figure 1.8 (taken from fig. 4.5.2, PCI [1989]) illustrates schematically solutions for different configurations of precast concrete units. (a) represents a typical (floor to floor) wall unit. (b) is a unit with a width less than six to eight feet, or narrow enough to disregard the horizontal restraint of the load supports. (c) shows a unit of such width that two intermediate lateral connections have been utilized.

"The designer should provide simple and direct load transfer paths through the connections and ductility within the connections. This will reduce the sensitivity of the connection and the necessity to precisely calculate loads and forces from, for example, volume changes and building frame distortions. The number of load transfer points should be kept to a practical minimum. It is highly desirable that no more than two connections per panel be used to transfer gravity loads, unless all are designed to carry substantially greater but indeterminate loads. Regardless, the bearing points should all occur at the same level. Load transfer should always be as direct as possible.

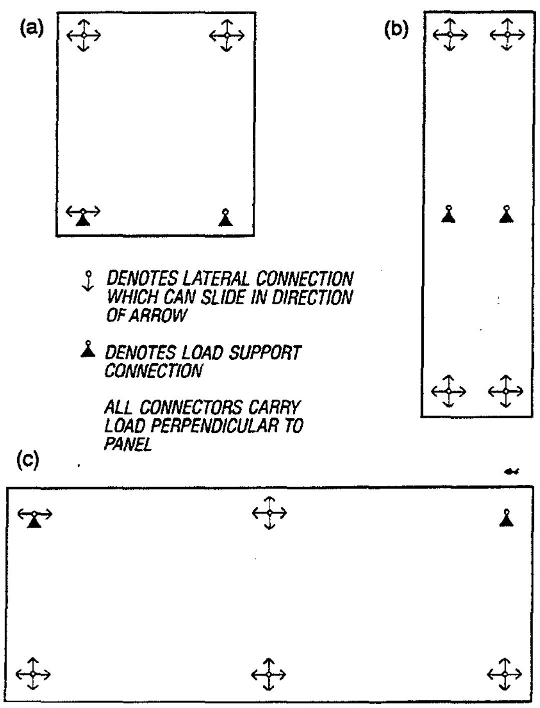


Figure 1.8. Connections for cladding panels in low seismic areas (from PCI [1989]).

"The impact loads associated with handling and setting precast concrete units may double the dead load used in the design of a connection. The magnitude of the impact loading is dependent upon the methods and controls of hoisting and the vulnerability of the connection (or its anchorage) to damage from impact loads. Where connections are designed for loads equal to or exceeding the impact loads, the requirements for impact have been automatically satisfied.

"In High Seismic Zones, the most common application of architectural precast concrete is a non-loadbearing cladding. The *Uniform Building Code* requires that 'precast or prefabricated non-bearing, non-shear wall panels or similar elements which are attached to or enclose the exterior shall be designed to resist the (inertial) forces and shall accommodate movements of the structure resulting from lateral forces or temperature changes.' The force requirements often overshadow the importance of allowing for moisture and thermal movement. Panels typically have two rigid load-bearing connections with volume change relief provided only by the ductility of the connections, and two or more tie-back connections with full freedom of movement in the plane of the panels.

"Ductility may be described as the ability of a material in the connection to stretch or 'give a little' when overloaded, without failing and causing resultant additional overstresses within the supporting structure. Connections should be designed such that if they were to yield, they would do so in a ductile manner, without loss of load-carrying capacity.

"Connections and joints between panels should be designed to accommodate the movement of the structure under seismic action. Connections which permit movement in the plane of the panel for story drift by bending of steel, properly designed sliding connections using slotted or over-sized holes, or other methods providing equivalent movement and ductility are also permissible. Story drift is defined as the relative movement of one story with respect to the stories immediately above or below. Between points of the connection, non-loadbearing panels should be separated from the building frame to avoid contact under seismic action. Story drift must be considered when determining panel joint locations and sizes, as well as connection locations and types.

"The *Uniform Building Code* requires allowance for 'story drift,' This required allowance can be 2 in. or more from one floor to the next and may present a greater challenge to the designer than the forces. This (*UBC*) requirement is in anticipation of frame yielding to absorb energy. The isolation is achieved using slots or (more often) long rods which flex. The rods must be designed to carry tension and compression in addition to the induced flexural stresses. In the case of floor to floor wall panels, the panel is usually rigidly fixed to and moves with the floor beam nearest the panel bottom (see fig. 1.9, taken from fig. 4.5.3a, PCI [1989]). In this case, the upper attachments become isolation connections and prevent the building movement forces from being transmitted to the panel, thus the panel translates with the load supporting beam. Some designers prefer to support the panels at the top and put the isolation connections at the bottom.

"Spandrel panels usually have the loadbearing connections at the top of the floor beam with the tie-back (also known as the push-pull or lateral or stay) connections located and attached to bottom of the same floor beam (see fig. 1.9, taken from fig. 4.5.3b, PCI [1989]). In this instance, the tie-backs are not affected by story drift since the top and bottom of the floor beam move together.

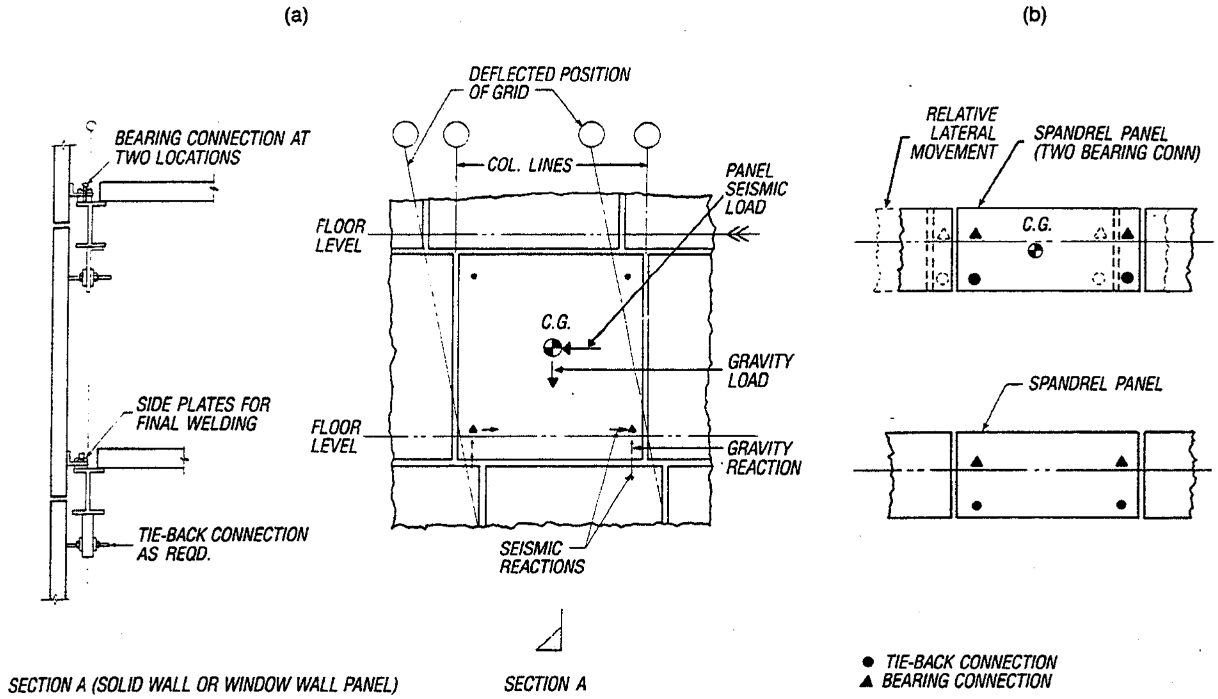


Figure 1.9. Panel connection concepts (from PCI [1989]).

"If the panel or column cover is narrow, the connection system is sometimes chosen to have both the top and bottom of the panel move with their respective floors and force the panel to rotate or rock up on one of the two loadbearing connections (see fig. 1.10, taken from fig. 4.5.4a, PCI [1989]). Since the movement occurs in both directions, each loadbearing connection must have the capacity to carry the full weight of the element without becoming tied down. Vertical movement, such as allowed with slots, must not be restricted as the panel rocks back and forth.

"The connection system determines panel movement. In figure 1.10 (taken from fig. 4.5.4a, PCI [1989]), seismic reactions at top together with 'lift off' allowance of bottom connections allow a panel to rotate with its entire weight being carried on one lower connection. In figure 1.10 (taken from fig. 4.5.4b, PCI [1989]), all vertical and inplane horizontal loads are carried near the center of gravity with connectors that keep it plumb and make it translate with connected floor. The upper and lower tie-backs must tolerate the drift.

"These movement capabilities must not be compromised with the need for adequate production and erection tolerances. If tolerances were $\pm^1/2$ in. and drift allowance was ±1 in., a slot length of 3 in. plus the bolt diameter would be required.

"It is essential that the types of movement (e.g., translation or rotation) be studied and coordinated not only with the connection system but the the wall's joint locations and joint widths.

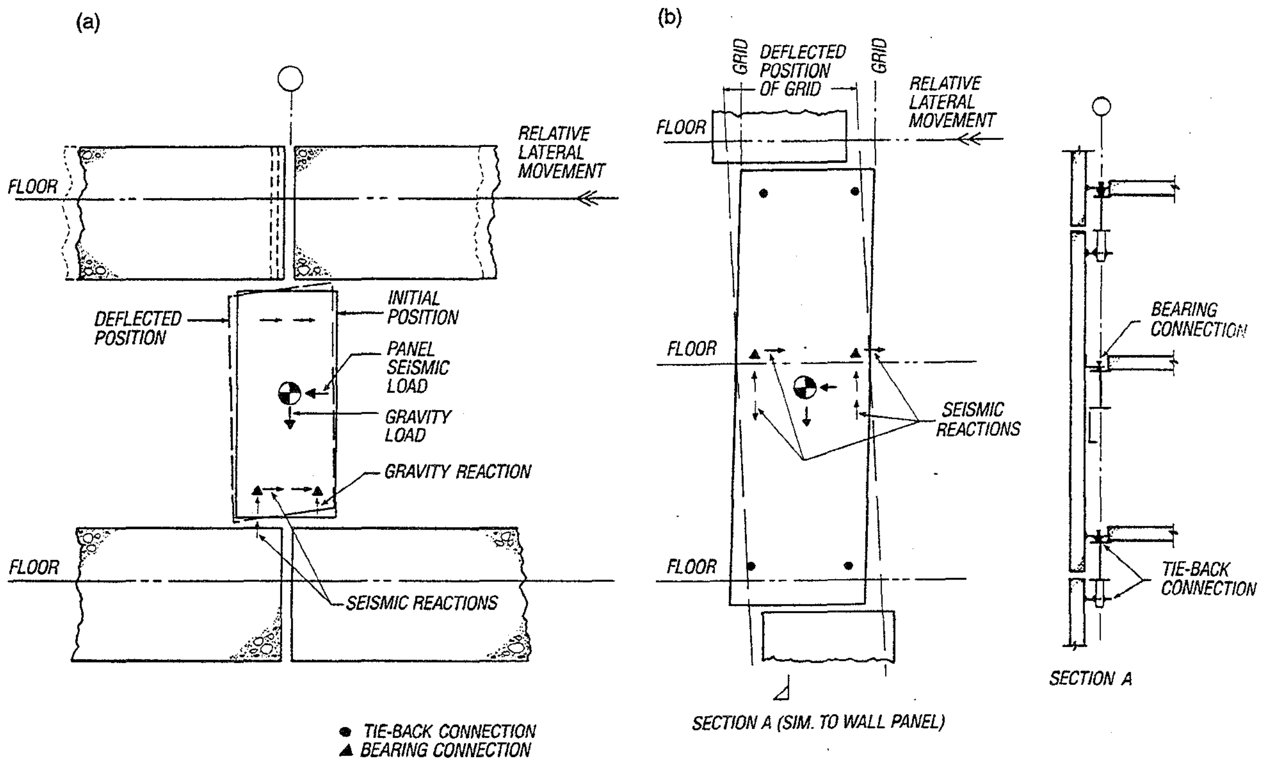


Figure 1.10. Tall/narrow units (from PCI [1989]).

For example, if a rotating column cover occurs between translating spandrel panels, the joint width must accommodate the amount of rotation that would occur in their common height. Such considerations may govern the connection system or the wall's joint locations.

"For seismic forces, the *Uniform Building Code* required that the body of the connector be designed for a force equal to $1^1/3$ times the required panel force and that the body be ductile. The code requires that all fasteners be designed for four times the required panel force. The anchorage to the concrete is required to engage the reinforcing steel in such a way as to distribute forces to the concrete and/or reinforcement and avert sudden or localized failure. The *Code* does recognize the advantage of this in calculating anchor strength. The engagement details are left to the designer. Since the force distribution philosophy is critical to seismic design and performance, it leads many designers to specify confining hoops (such as UCS5, fig. 1.11 taken from fig. 4.5.64, PCI [1989]), deformed bar anchors, or long reinforcing bars welded to plates, rather than headed studs or inserts. With appropriate orientation, the reinforcing anchors will act in tension with optimum efficiency. It studs are used and loaded near the edge of the concrete panel, it is recommended that they be enclosed in sufficient reinforcing steel to carry the loads back into the panel so a sudden tensile failure mode in the concrete is averted.

Design

- has very high load capacity when it engages and confines panel reinforcement
- good for dynamic loads, i.e. seismic
- size variability makes it adaptable to many panel configurations

Production

- expensive fabrication but alternates for equal capacity may be more costly

Variations

- bearing lug may be desirable to reduce shear on loop anchors

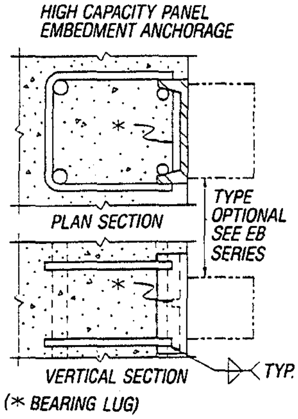


Figure 1.11. Unique conditions & solutions (UCS5) (from PCI [1989]).

"When possible, it is advantageous to arrange concrete anchor studs so that the ones that carry tension due to gravity do not have to carry tension due to seismic forces." An example of this is given in PCI [1989], page 190.

"In many cases, the wall panels are sufficiently outboard of the supporting frame, to require either outriggers off the beam or long panel brackets... For seismic forces in the plane of the panel, anchorage of the longer panel brackets to the panel can be come quite cumbersome, since the forces must be combined with gravity." An example is given in PCI [1989], page 190.

"The panel shown in figure 1.12 (taken from fig. 4.5.5, PCI [1989]) illustrates load support connections for medium size units in earthquake Zone 3. This is an example of precast concrete units serving only as rain barriers, with the exterior cast-in-place shear wall serving as an airseal. The load supports were placed in recesses at the windows, making them readily accessible (see fig. 1.13, taken from fig. 4.5.6, PCI [1989]). Following panel installation, these recesses were concreted to complete the exterior airseal and fireproofing.

"It is important to coordinate the design and detailing of connections with other functions, such as production, erection, tolerances, and joints." Further information is given in PCI [1989].

Figure 1.12. Load support connections for medium size units, seismic Zone 3 (from PCI [1989]).

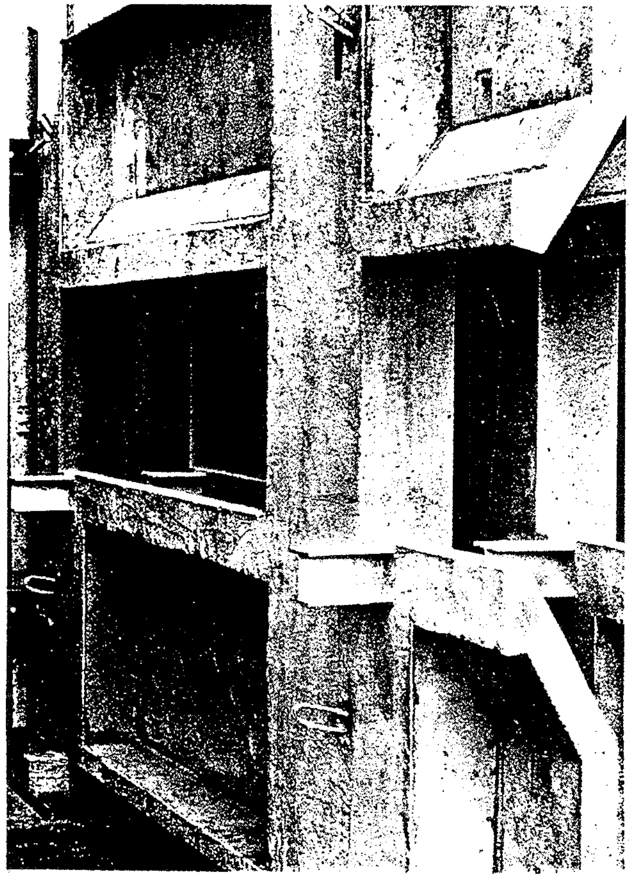
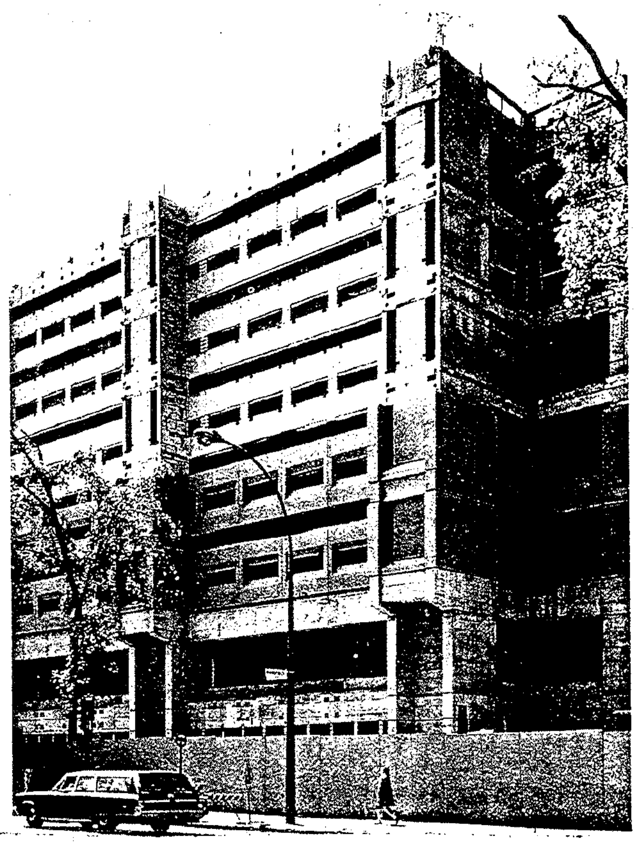


Figure 1.13. Load supports placed in recesses at the windows (from PCI [1989]).



In Iverson [1989], the author discusses concrete cladding connections in seismically active regions. He reviewed the general practice in the design of cladding connections in U.S. seismic Zones 3 and 4. Excerpts from his paper are presented herein.

Iverson commented that "The success of cladding... has been related to the acceptance of the cladding connections both in meeting strength requirements and just as importantly meeting ductility requirements. Often the response to establishing ductility has been to increase strength requirements to a level where only elastic action will probably occur.

"Current seismic design is based on using pseudo-static forces to size members and connections. These forces are recognized as being well below the actual forces the structure will experience in the maximum design earthquake, and hence the process assumes that the structure will experience inelastic movement, i.e., the concrete will fracture or the steel will yield at high stress locations. The structure is saved from collapse in this inelastic movement by its ductility, which is obtained by careful attention to details. This means that heavy secondary reinforcing will be required at locations where yield hinges will form.

"A further complication that often complicates design of the cladding is that much of the construction utilizes steel frames. The steel frame is very flexible in comparison to the rigid concrete panel and the connection must be designed to accommodate the relative movement expected in a large seismic event between the frame and still support the concrete panel." Korista [1989] also expressed the same concern by stating, "...the thorough understand of the force system generation and the deformation response behavior of each adjacent system is an absolute necessity in understanding the critical deformation compatibility issues between (cladding and framing) systems."

Iverson [1989] continued by posing a question: "What is the primary difference in the seismic design of cladding from that in non-seismic regions? Put simply, there is one more significant horizontal force that must be considered and the movements mentioned above must be accommodated. But the kicker is the empirical requirements to maintain ductility. An example is the loads permitted on headed anchor studs. As recently as 1987, and probably still today, the City of San Francisco requires that studs be designed to Table 26-G in the *UBC*. This table was developed just after World War II and applied to embedded bolts. Reductions from comparable stud values in the PCI handbook, even when ultimate strength factors are considered, [are] on the order of 4 or much more. The *UBC* table values may be exceeded, but physical testing is required and the extent of hence cost of this work is strictly based on the individual Building Official's requirements and are usually quite expensive. Most precast manufacturers use the table values rather than gamble on the costs of the testing.

"One clear conclusion of any analysis of cladding connections is that a considered, orderly, clearly reported research program is essential if widespread acceptance of reasonable connections is to be expected.

"COMMON CONNECTIONS: The movement criteria due to flexing in the steel frame have a considerable effect on the connection. The present maximum allowable story drift is 0.005 of the story height, based on low pseudo-static design forces." [*Note:* This is a serviceability drift limit for which the framing is to remain essentially elastic.] "Some idea of the magnitude of the problem can be realized by using this maximum limit in typical conditions. Assuming a 12 ft. story height this gives a story drift of: 0.005x12x12 = 0.72 in."

"The *UBC* further requires that connections accommodate from 3 or 4.5 times this story drift, depending on the building type. Again, this is essentially a ductility requirement. So the connector theoretically must move sideways up to 2 or 3 inches. The most common solution to this problem is to provide two types of connectors on the panel, those for gravity loads that are free to slide sideways to accommodate drift movement and those to resist the smaller horizontal forces and are flexible in the opposite direction and will deflect to accommodate movement... The horizontal seismic forces on the panel are resisted by the threaded rod, which must deform to accommodate the story drift requirements. The heavy, vertical gravity loads are resisted by the cantilevered tube, which slides during the earthquake drift movement.

"Another common solution is to use bolted connections with slotted holes and this works best in the stiffer supporting structures, where drift is smaller.

"In some recent designs, a thin triangular [trapezoidal?] plate is used at the center of the panel and [which is] to welded to the beam, and supports the horizontal seismic forces of the panel. Since it is near the center of the panel, it can accommodate the drift deformations by twisting rather the bending and of course if kept thin to accomplish this without rupture... The threaded 'push-pull' anchor rods must still be used to provide overall torsional stability and allow panel alignment, but their size is considerably reduced.

"The heaviest load is still the vertical gravity load and cantilevered tubes seem to be increasingly used for this function, particularly as larger panels are used. The tube of course is often coupled with a bolt to allow field leveling of the panel to accommodate tolerances.

"Many other combinations of materials and systems are used. Angles and channels are often used for the cantilevered gravity support and in situations where sideways story drifts are limited, flexible steel plates are welded on the sides of the tube to support the horizontal seismic forces. These plates are kept long and thin to allow horizontal movement of the gravity connection.

"One of the problems that is of some concern in this type of work is the 'secondary steel' that carries the panel load back to the main supporting frame of the building. The design of this material most commonly falls to the panel detailer and the concern is the checking of the main steel where the concentrated seismic loads from the cladding are delivered to them. These members also support the overall seismic and building gravity loads and these taken in combination with cladding loads may lead to instability in the frame members and one often wonders if proper attention is devoted to this problem... This problem is clearly one that must be evaluated by the Engineer of Record, since usually he alone has information on the frame loads.

The interested reader is referred to Iverson [1989] for figures containing photographs of connections. [These photos did not reproduce well enough to include here.] Drawings of these connections can be found in PCI [1989].

Hegle [1989], a precast cladding panel producer, described "design considerations which should be followed to provide for the economical attachment of precast concrete cladding to a building structure. Panel configuration, production, transportation, erection, loading, and connection types are discussed for non-structural cladding."

He stated, "Architectural precast concrete cladding connections are generally designed to transfer cladding loads to the structure without affecting the response of the structure to vertical loads and lateral wind or seismic loads. Floor and roof members must be able to deflect and column drift must be accommodated without imposing loads on the cladding connections from the structure... This can be accomplished by identifying and providing for the interrelated architectural, structural and cost requirements of the building design."

He presented sections on cladding panel configuration, panel connection design, and connection types and loads. All sections, due to their importance, are given here in full. Some of the information is an extension of what is given in PCI [1989], with an emphasis on U.S. seismic Zone 4.

In the section on cladding panel configuration, he stated, "The architectural design of a precast building facade is usually enhanced by the use of real and false joints to create a pattern. The location of real joints between individual cladding panels must be carefully chosen.

"Generally, the joint will create three types of of panels: story height wall panels, horizontal spandrel panels, and vertical column cover type of panels" (see fig. 1.14 from fig. 1 in the paper).

"First, the joints must permit the individual panels to move as required to follow the building drift under lateral loading. Each story should have at least one real horizontal joint continuous all the way around the building. This will permit the panels attached to one floor to move with that floor's drift relative to the panels above and below them which must move with their floor's drift.

"Next, we must consider the location, size and capacity of the building structure to support the loads from the cladding panel connections. Whenever possible, panel bearing connections should be located at the building columns. Column supported connections are more economical than beam bearing connections and provide stiffer resistance to the panel eccentric loads. Real vertical joints at column lines thus offer an advantage.

"The overall size and weight of each individual panel can also be limited by the capacity of the local production facility, truck transportation legal limits, truck and crane access around the structure, and the available crane capacity.

In the section on connection types and loads, he stated, "Cladding panel connections must transfer gravity, wind and seismic loads from the panels to the structure. Generally they can be divided into three types: bearing connections (shown as solid triangles in fig. 1.14), lateral load connections (shown as solid circles), and shear load connections (shown as thick horizontal lines).

"Each panel may have one or two bearing connections, but never more than two. The panels are generally very stiff relative to the supporting structure so the use of more than two bearing points to support a panel creates unknown loads in each connection.

"Bearing connections transfer panel gravity loads, wind and seismic loads perpendicular to the panel, and may also transfer seismic loads parallel to the panel. They are generally located near the ends of the panels to provide a stable base during panel erection.

"Lateral load connections only transfer loads perpendicular to the panel. They are designed to permit the structure to move vertically and horizontally parallel to the panel while under perpendicular loading. They are located above or below the bearing connections and along the length of the panel as necessary to support the panel designed as a continuous beam for perpendicular loading.

"Shear load connections transfer loads in all horizontal directions while permitting the structure to move up and down behind the panel. They are located near the middle of wall and spandrel panels and at one end of vertical column cover panels.

In the section on panel connection design, he stated, "The configuration and design of each type of panel connection must consider a number of important characteristics. Providing a safe, economical solution to supporting cladding panels on a building frame requires that the connections be designed as follows:

1. "To transfer erection as well as final loads to the structure." (see paper for more detail)
2. "For ease of fabrication." (see paper for more detail)
3. "To accommodate building construction tolerances." (see paper for more detail)
4. "For economical panel erection... See figures 1.15 and 1.16 (taken from figs. 2 and 3 in the paper) for a bearing connection which only requires the placement of one bolt before the crane is released to hoist another panel. This type of connection also permits later movement of the panel in all directions for final alignment.
5. "To permit the structure to move: The connections must be capable of carrying their design loads while the structure is deflecting due to the gravity or lateral loading. This may be accomplished with slotted holes or bending of steel connection members. Two examples of lateral connections with this capability are illustrated in figures 1.17 and 1.18 (taken from figs. 4 and 5 in the paper).
6. "To fit within the architectural finish." (see paper for more detail)

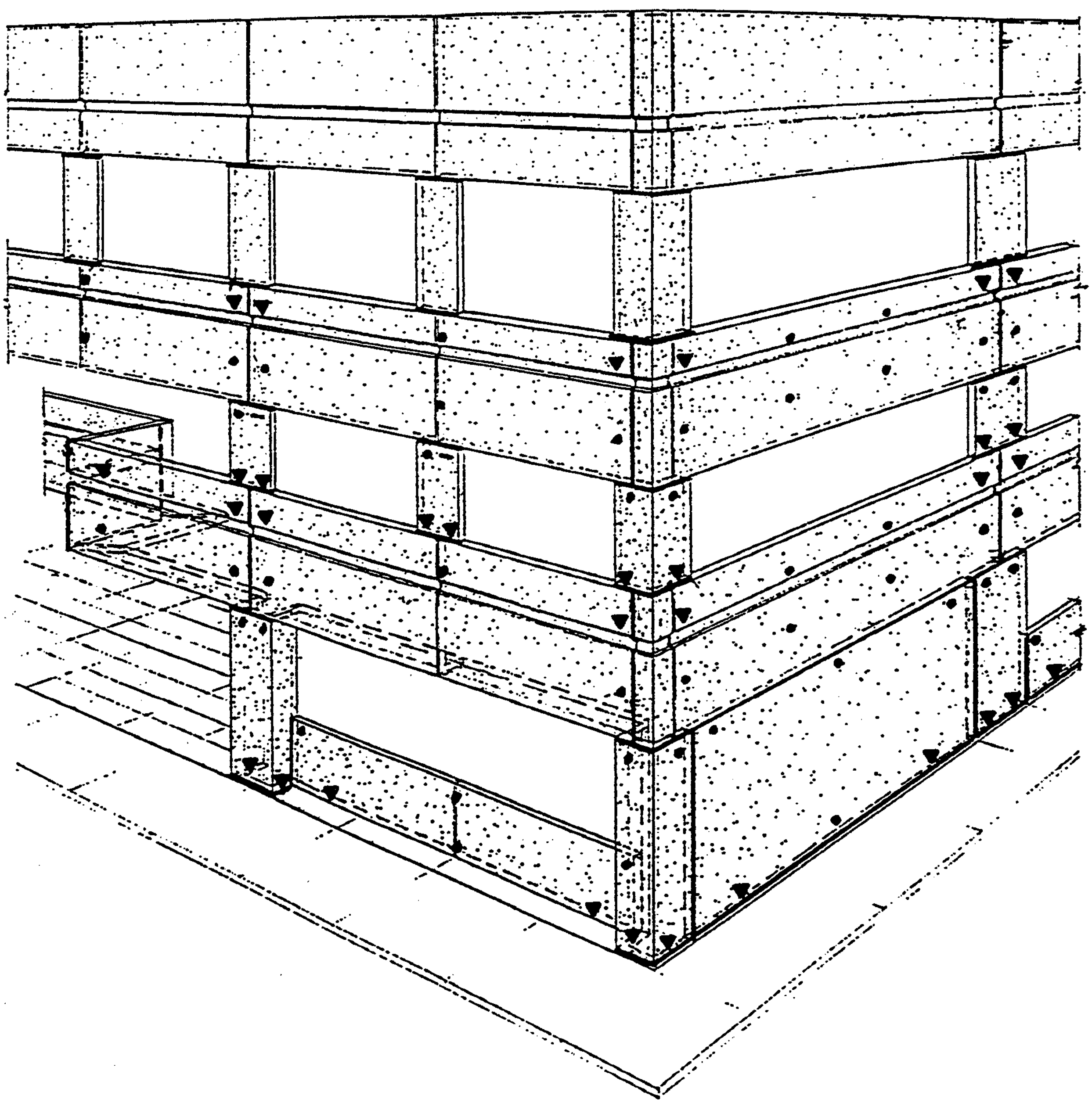


Figure 1.14. Typical architectural precast concrete panel building facade with real joints and panel connection locations shown (after Hegle [1989]).

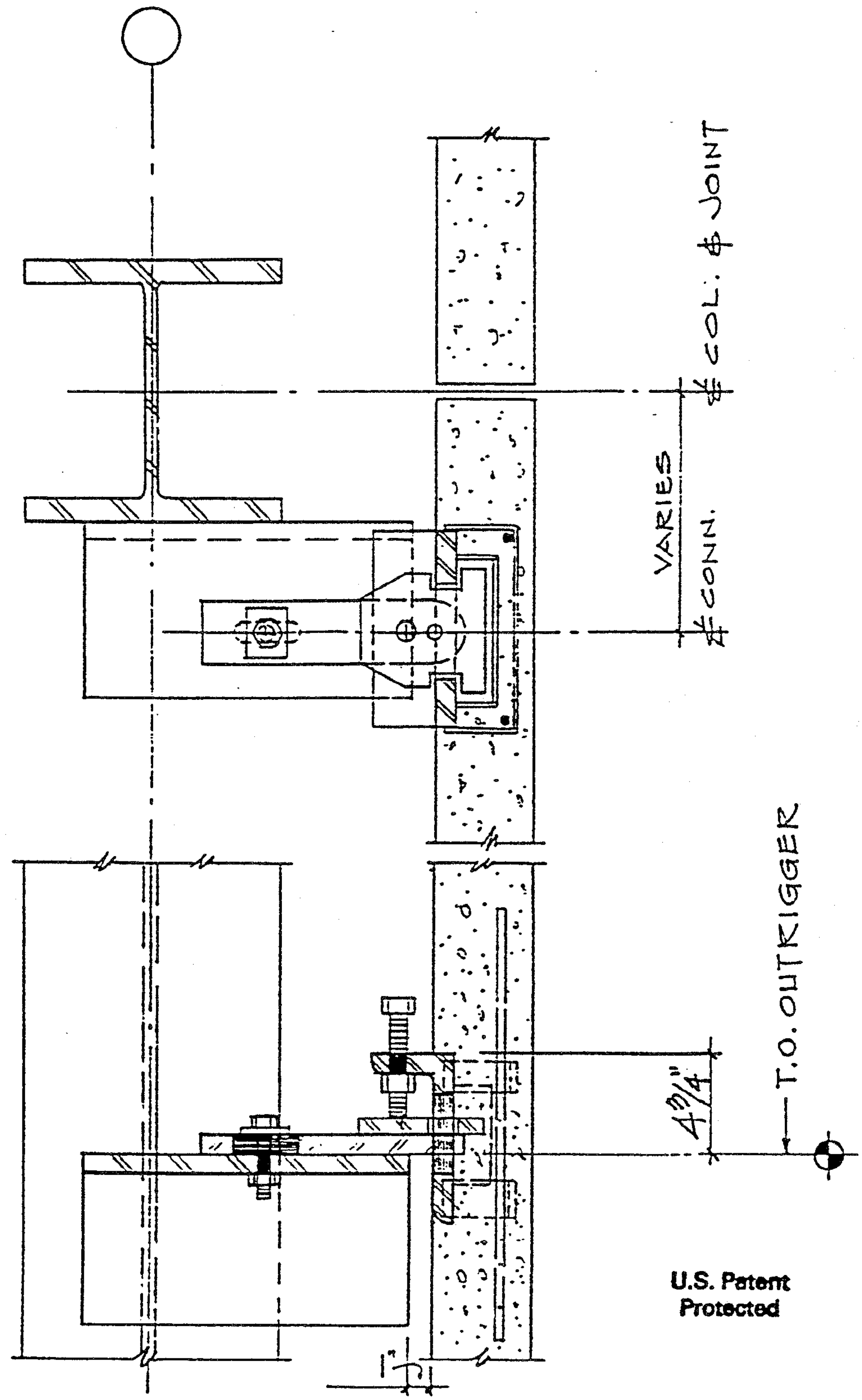


Figure 1.15. Bearing connection at column (after Hegle [1989]).

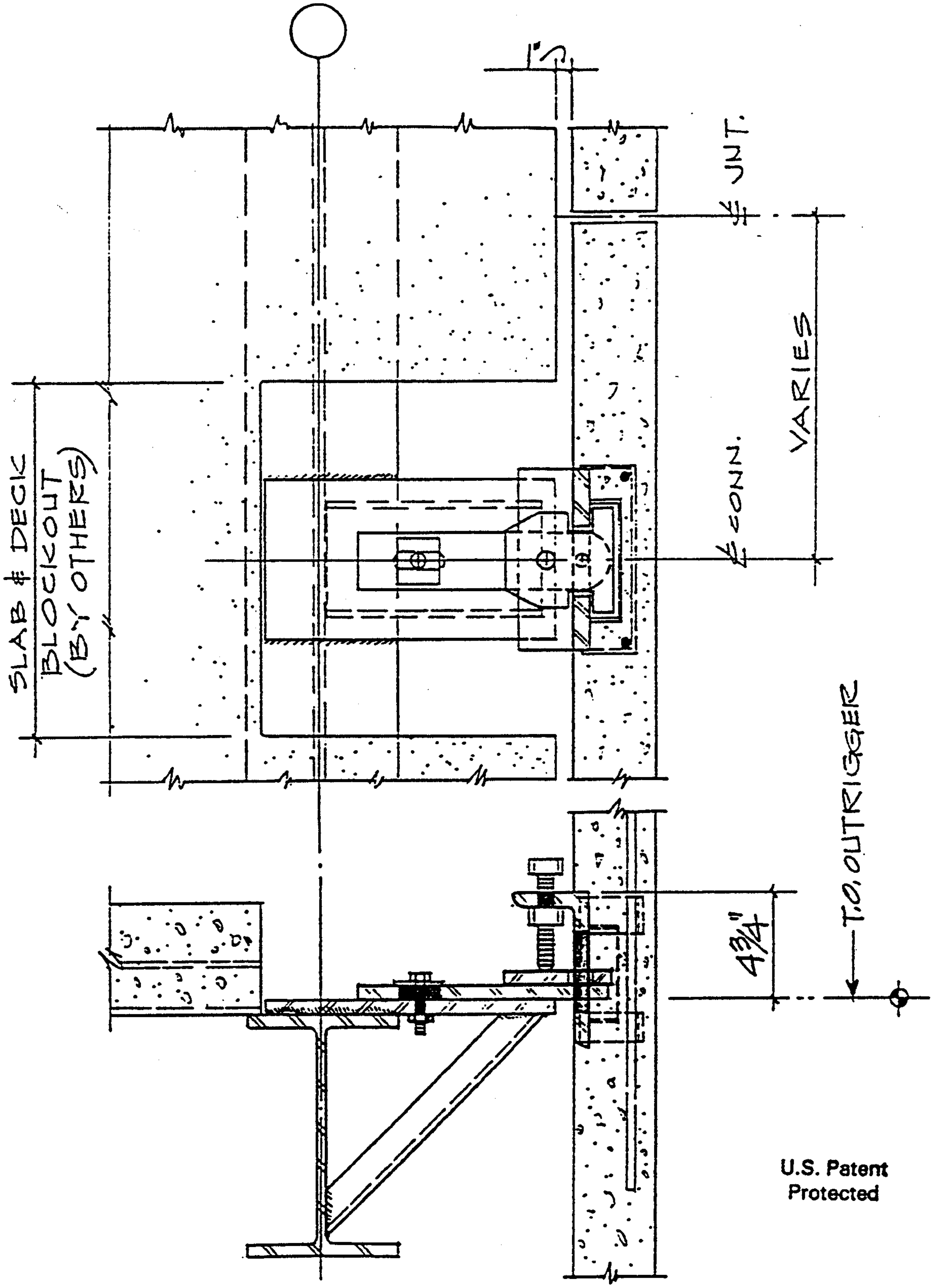


Figure 1.16. Bearing connection at beam (after Hegle [1989]).

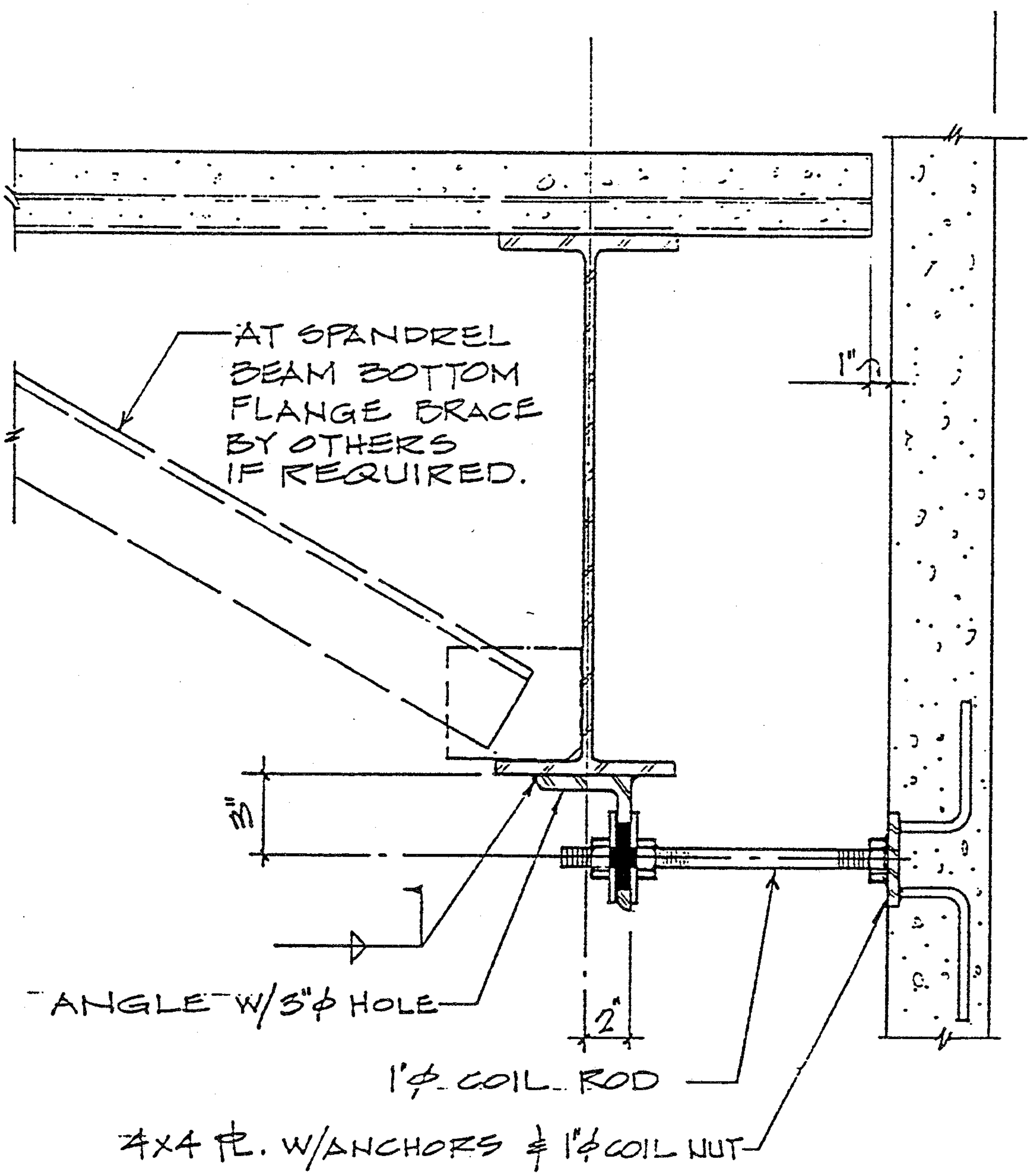


Figure 1.17. Lateral connection to beam or column (after Hegle [1989]).

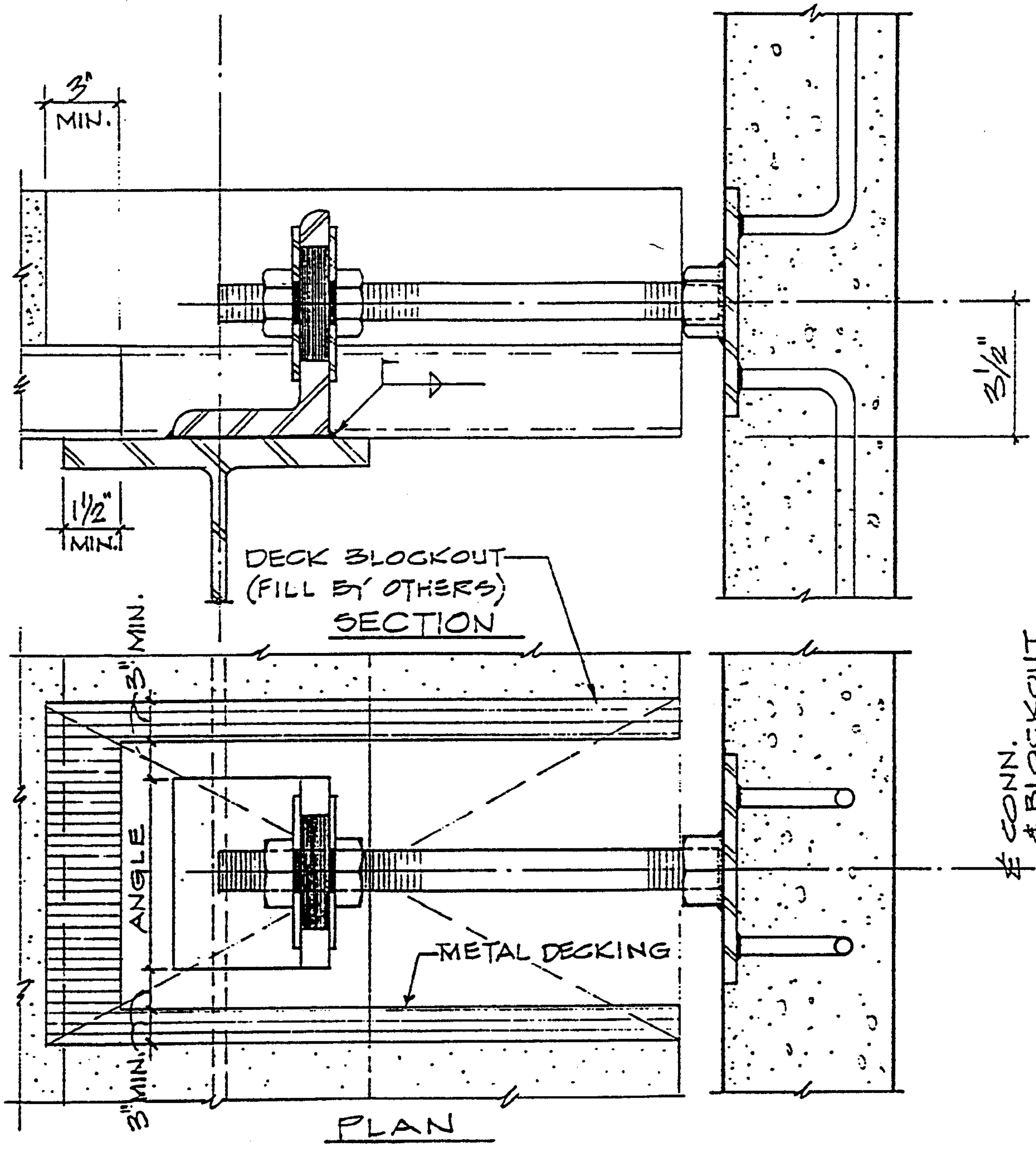


Figure 1.18. Lateral connection to floor (after Hegle [1989]).

McCann [1991] presented comprehensive materials on architectural precast concrete cladding connections in the continuing education seminar for the Structural Engineers Association of Northern California. His paper included the following sections: (1) code requirements, including structural panels, future - participating panels, and non-structural cladding panels (for movement (drift), ductility, and strength); (2) system concepts including top bearing brackets, bottom bearing brackets, and intermediate support (for column and spandrel covers); (3) cooperative effort of design and construction team, including architectural domain (for window system and fire protection, and joint locations and widths), structural engineer of record (for support location and provisions), and panel supplier (for practical details); (4) bolt, weld, or grout (for temporary and/or final); (5) tolerance, clearance, and movement. (6) anchorage to concrete, including inserts, bolts, and studs, rebar and other shapes, and drill-ins - wet and dry; (7) bearing connections, including panel brackets and shear plates; (8) tie-backs (aka push-pull, stay, lateral), including slide, flex, or pivot, and receivers; (9) miscellaneous, and (10) glass fiber reinforced concrete, including material characteristics, and anchors and connectors.

McCann's figures are presented here as figures 1.19 to 1.40, including: (1) story drift; (2) panel inertia reactions; (3-4) bearing location effect; (5) spandrel connectors; (6) windowed wall - panel types; (7) pounding potential; (8) free translation; (9) load-reaction couple; (10) typical concrete anchors; (11) cast-in anchorage; (12) eccentric bearing brackets; (13) eccentric bearings; (14) shear plates - fixed; (15) shear plates with lift-off; (16) long tie-backs; (17) special tie-backs; (18) tie-back rod receivers; (19) oversize hole considerations; (20) plate washer tricks; (21) controlling anchorage loads; and (22) glass fiber reinforced concrete.

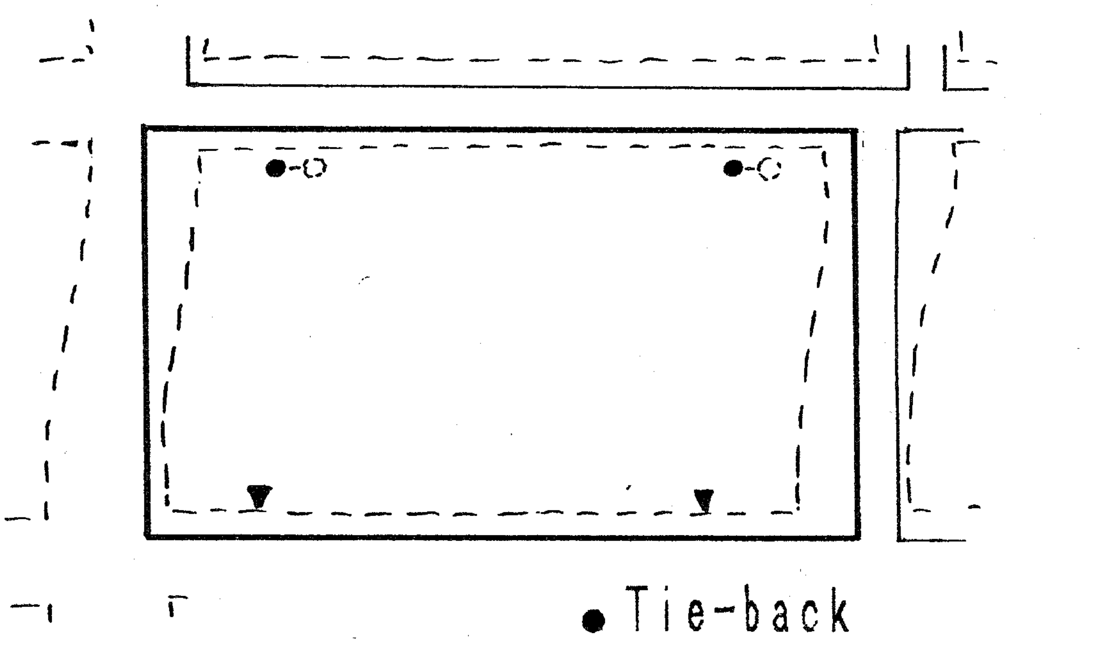


Figure 1.19. Story drift (from McCann [1991]).

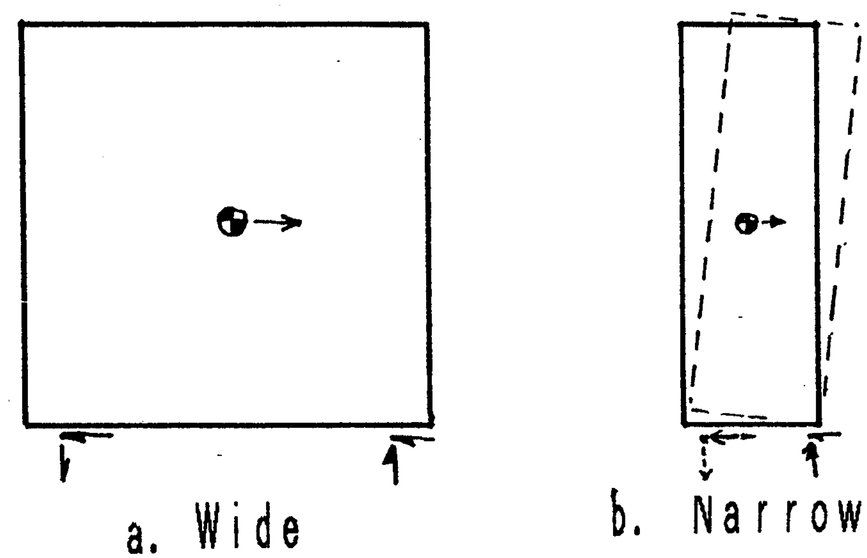


Figure 1.20. Panel inertia reactions (from McCann [1991]).

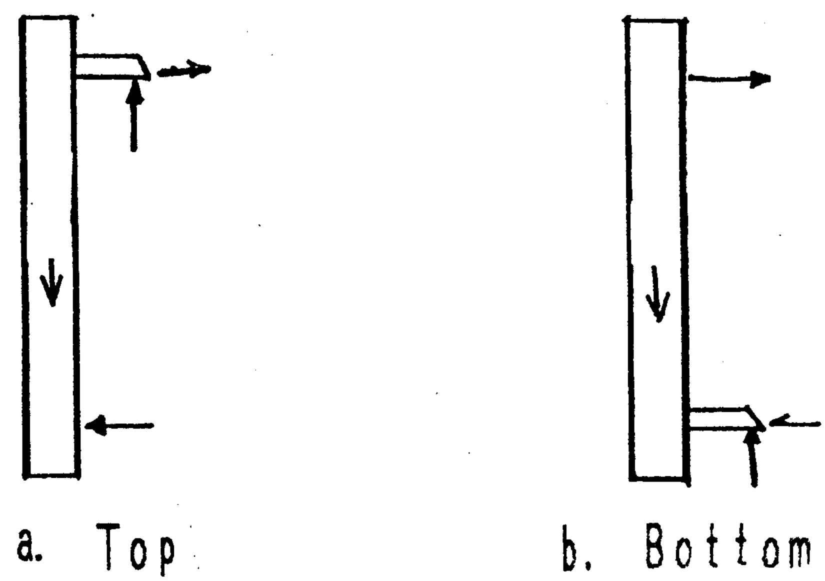


Figure 1.21. Bearing location effect (from McCann [1991]).

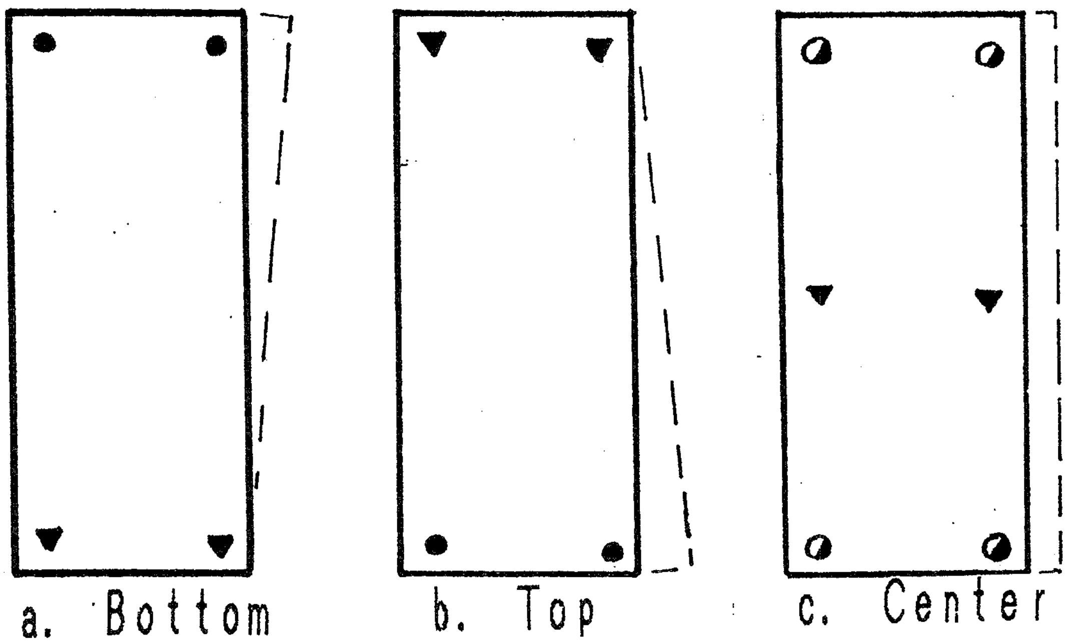


Figure 1.22. Bearing location effect (from McCann [1991]).

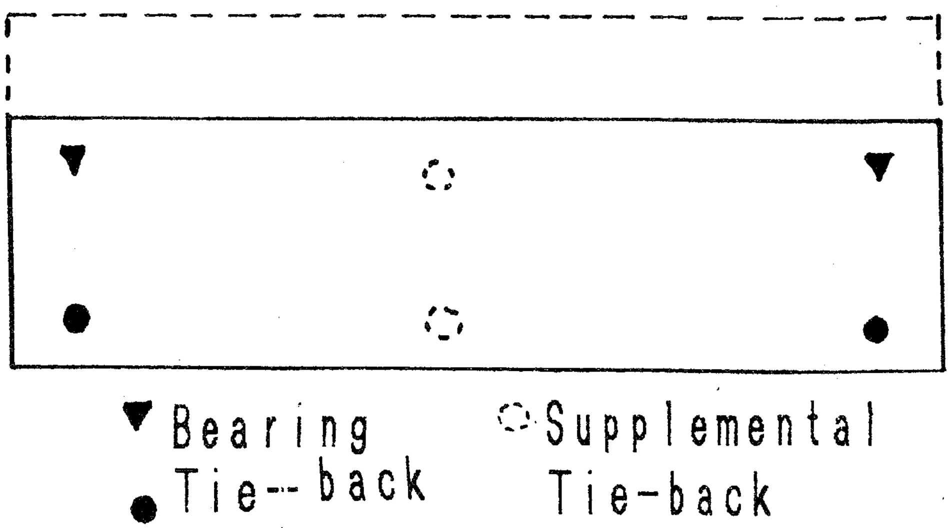


Figure 1.23. Spandrel connectors (from McCann [1991]).

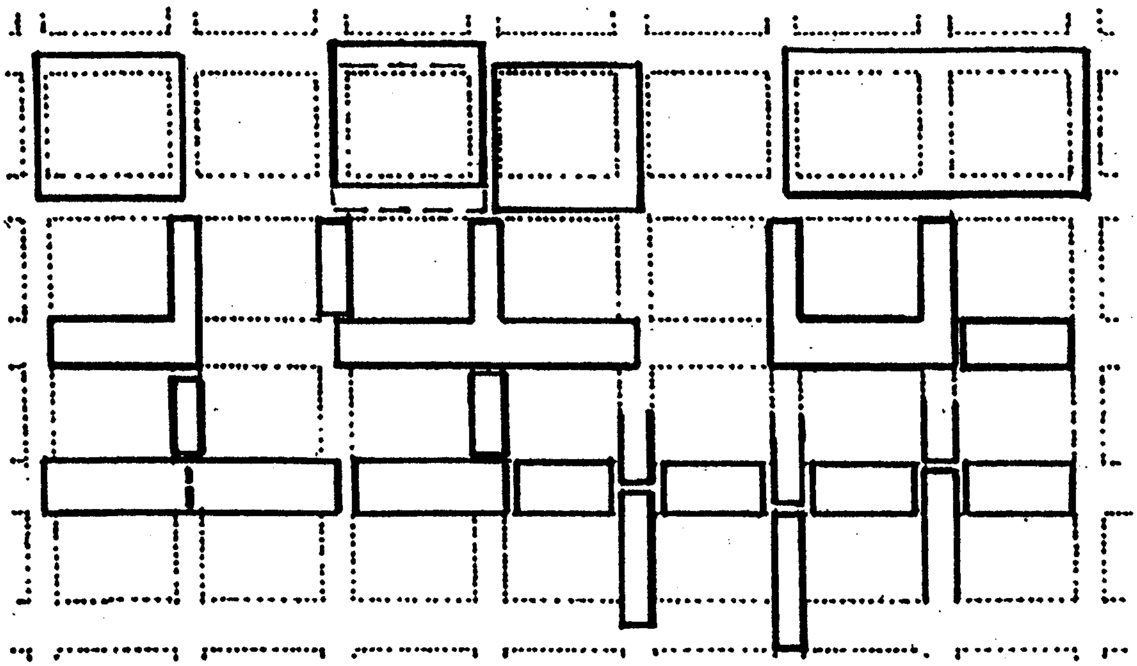


Figure 1.24. Windowed wall - panel types (from McCann [1991]).

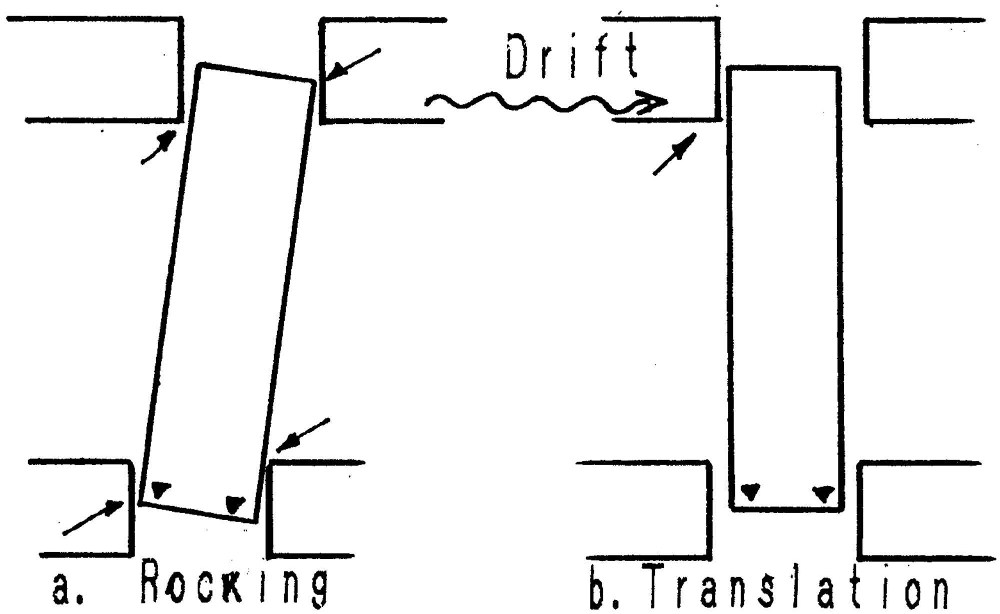


Figure 1.25. Pounding potential (from McCann [1991]).

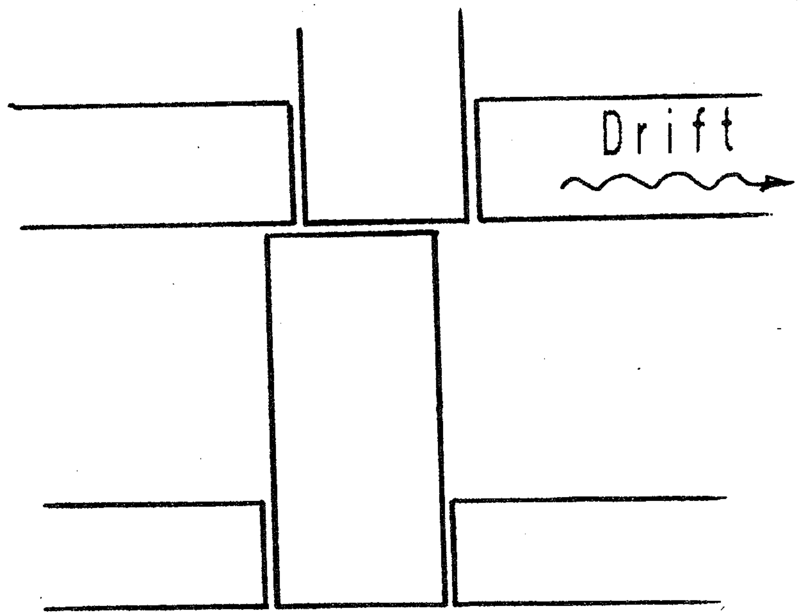


Figure 1.26. Free translation (from McCann [1991]).

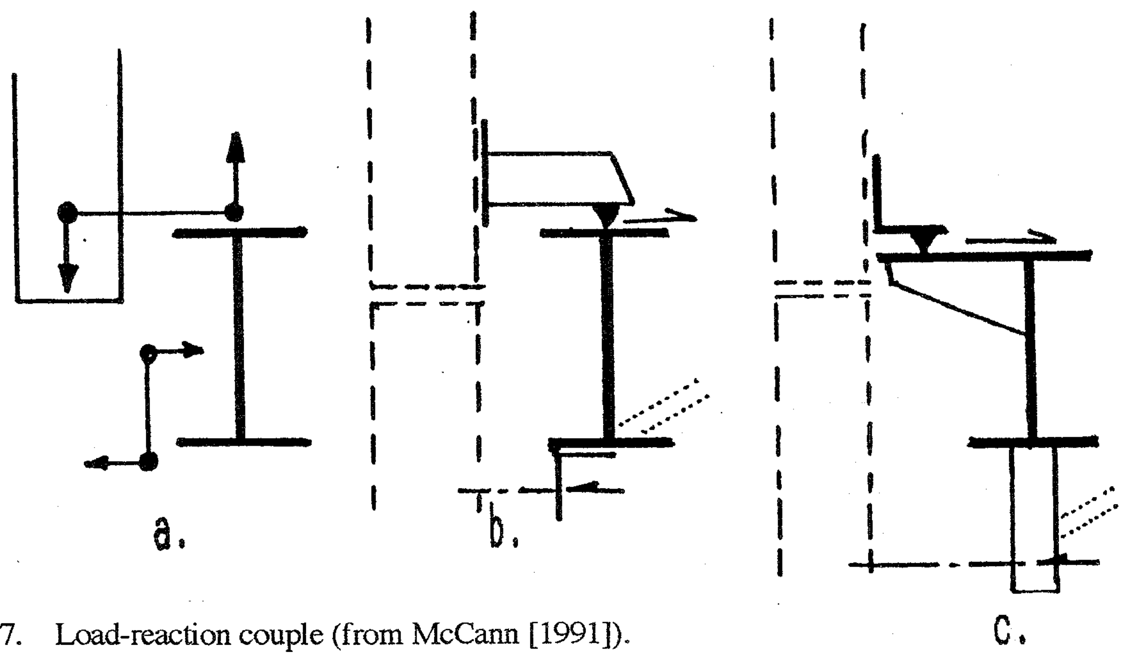


Figure 1.27. Load-reaction couple (from McCann [1991]).

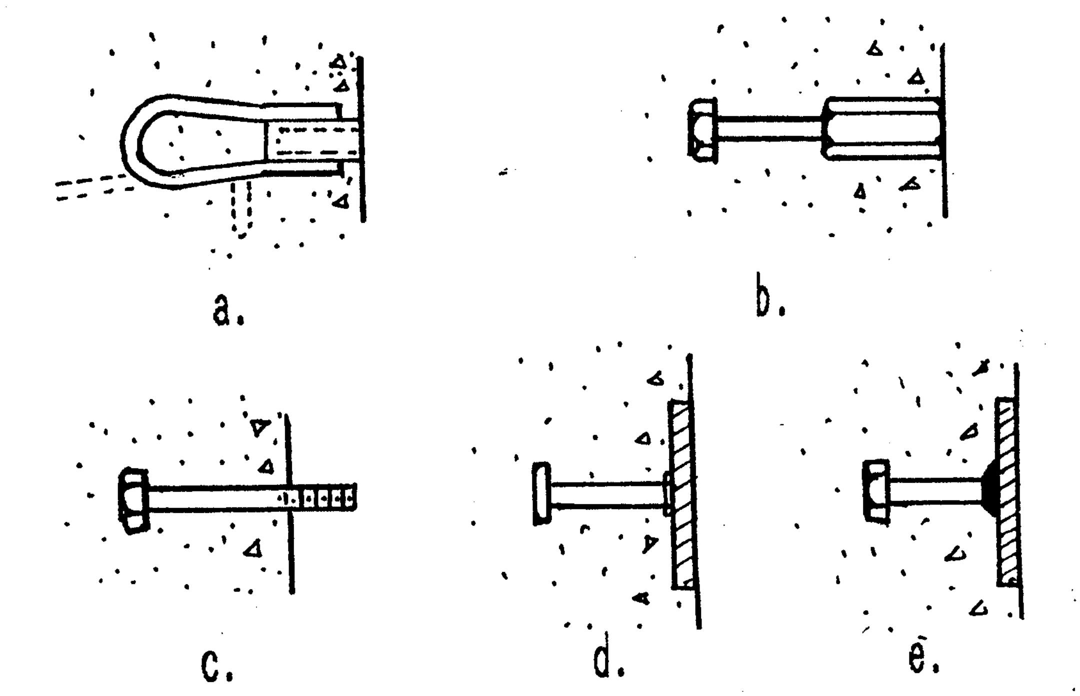


Figure 1.28. Typical concrete anchors (from McCann [1991]).

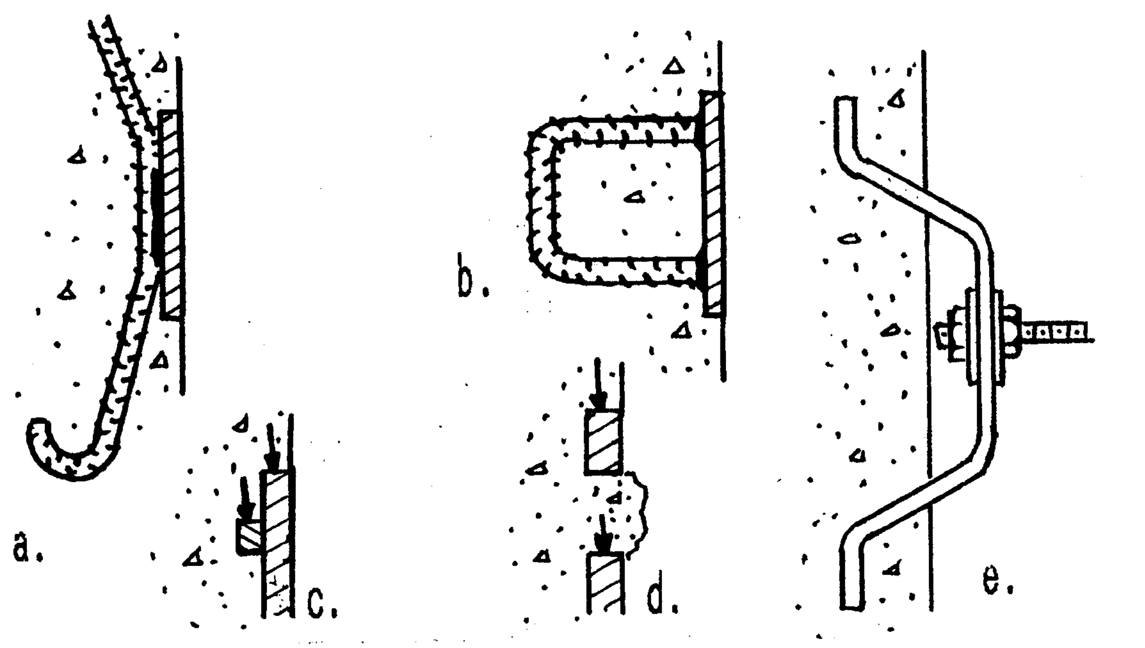


Figure 1.29. Cast-in anchorages (from McCann [1991]).

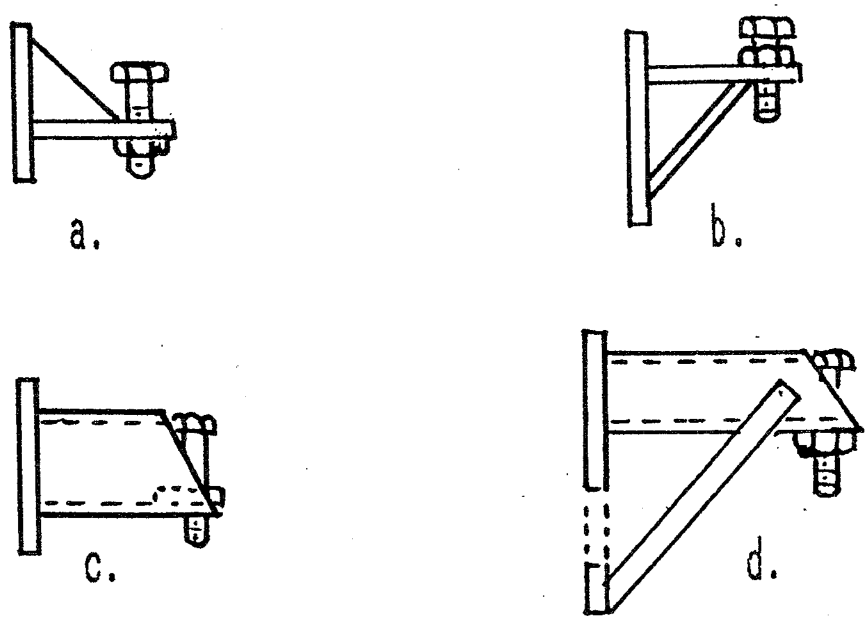


Figure 1.30. Eccentric bearing brackets (from McCann [1991]).

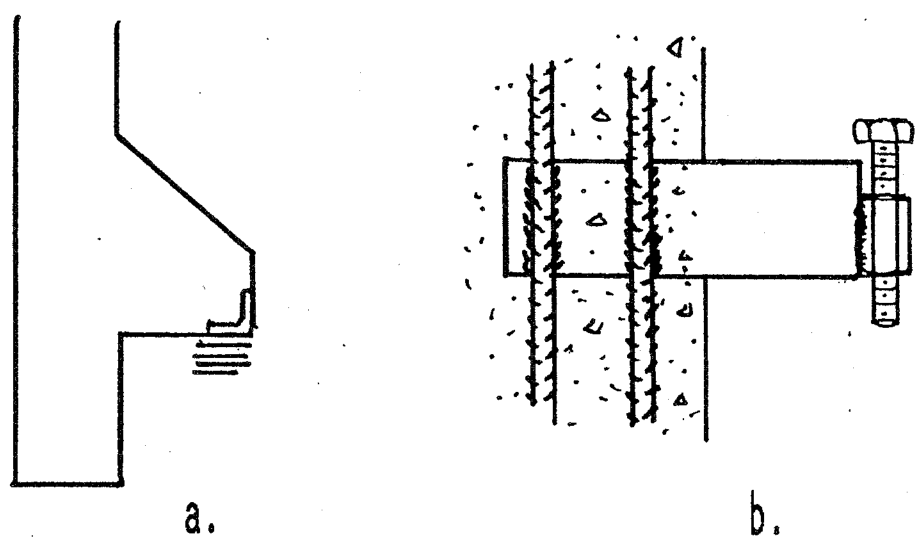


Figure 1.31. Eccentric bearings (from McCann [1991]).

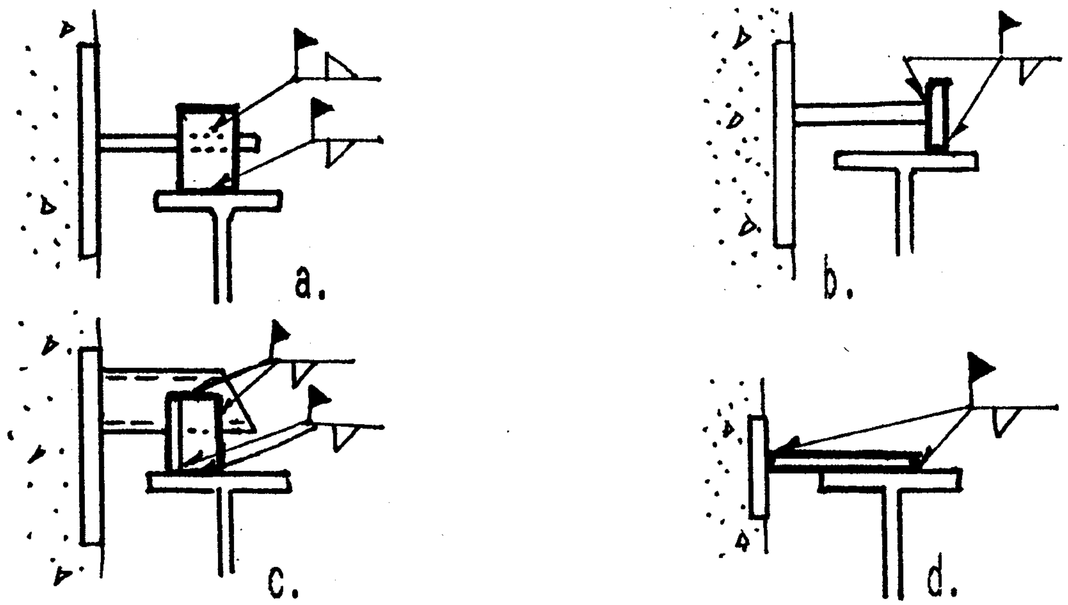


Figure 1.32. Shear plates - fixed (from McCann [1991]).

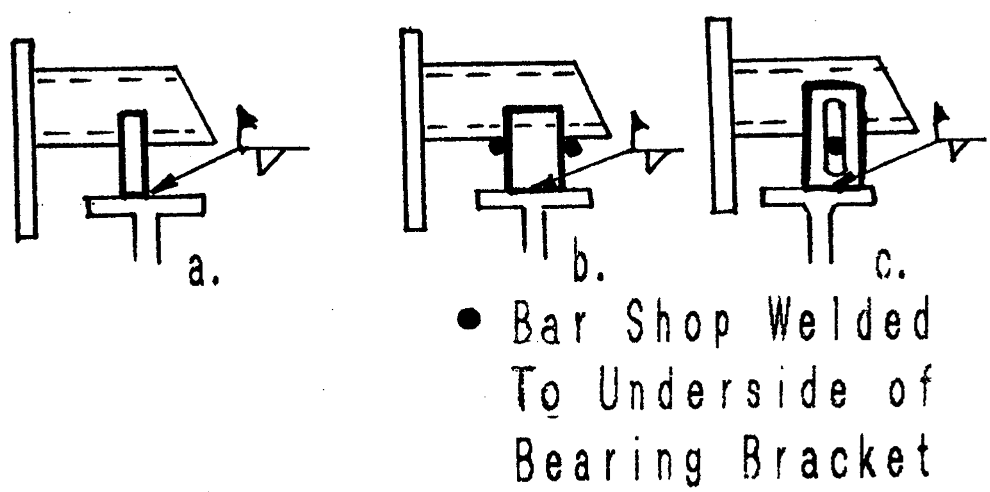


Figure 1.33. Shear plates with lift-off (from McCann [1991]).

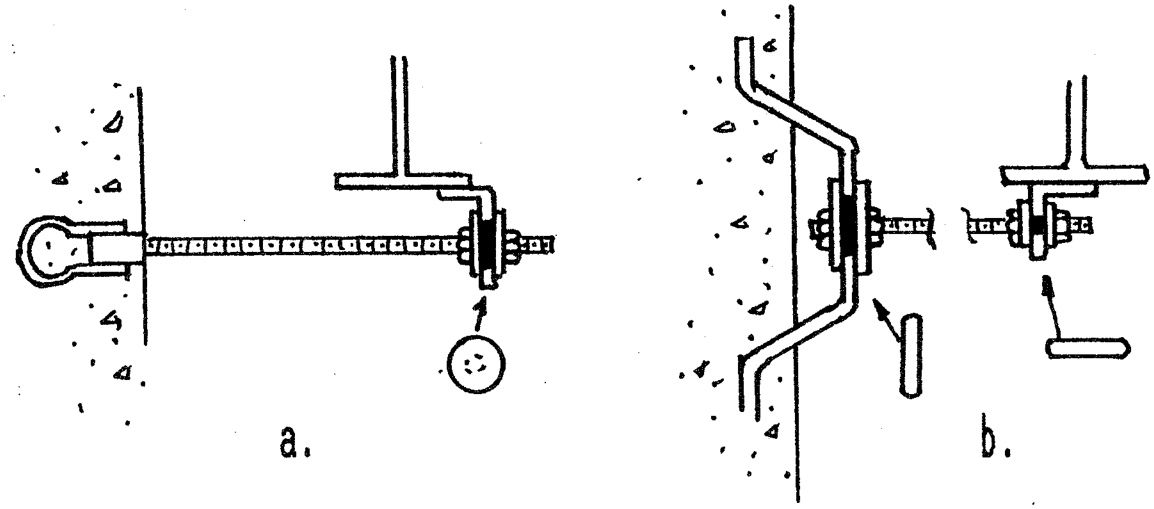


Figure 1.34. Long tie-backs (from McCann [1991]).

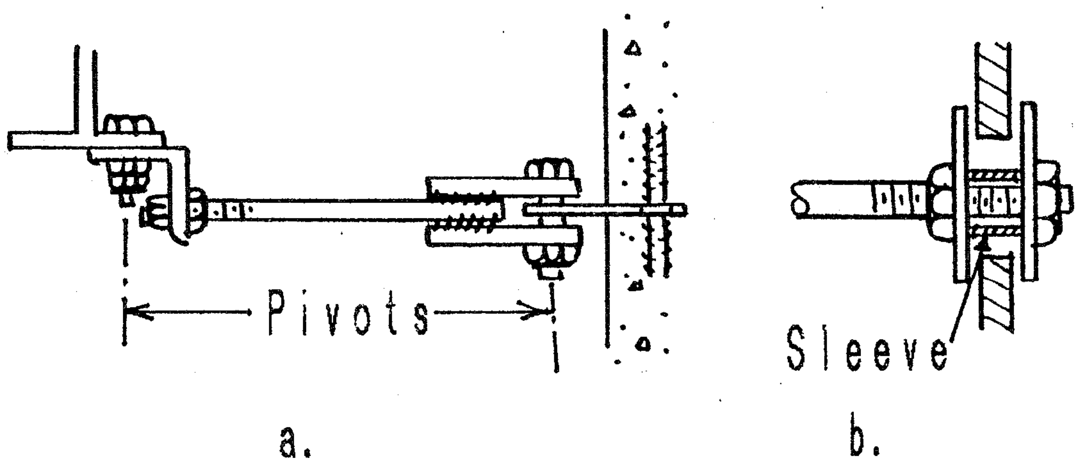


Figure 1.35. Special tie-backs (from McCann [1991]).

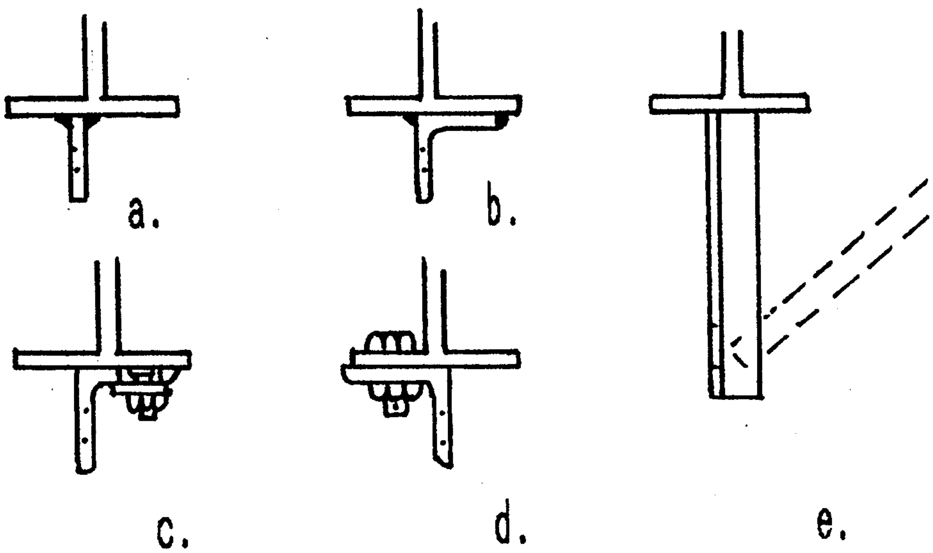


Figure 1.36. Tie-back rod receivers (from McCann [1991]).

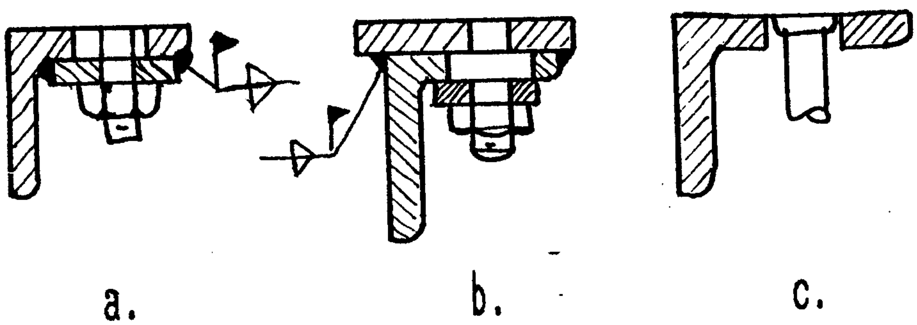


Figure 1.37. Oversize hole considerations (from McCann [1991]).

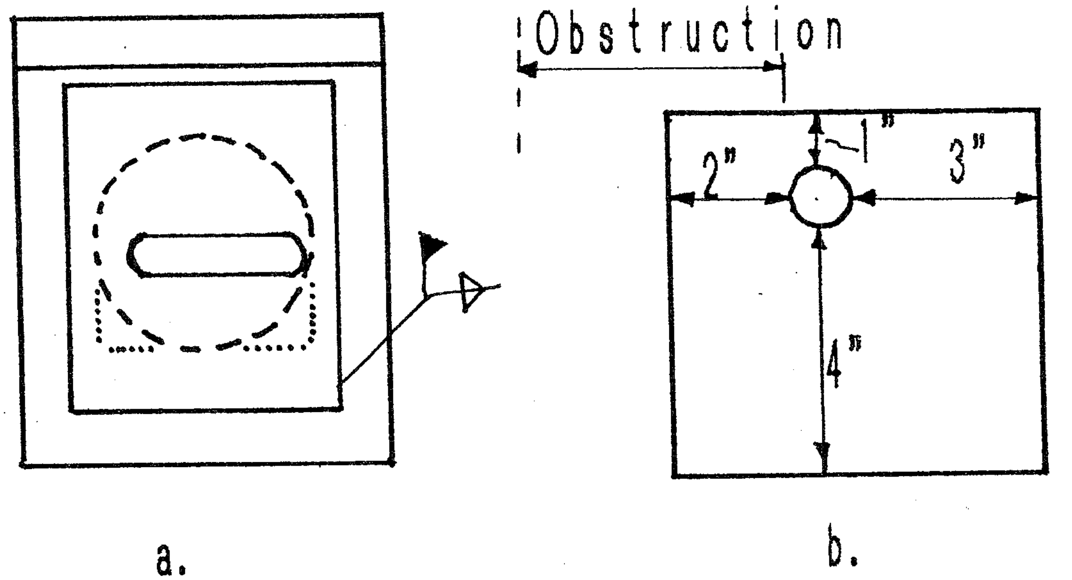


Figure 1.38. Plate washer tricks (from McCann [1991]).

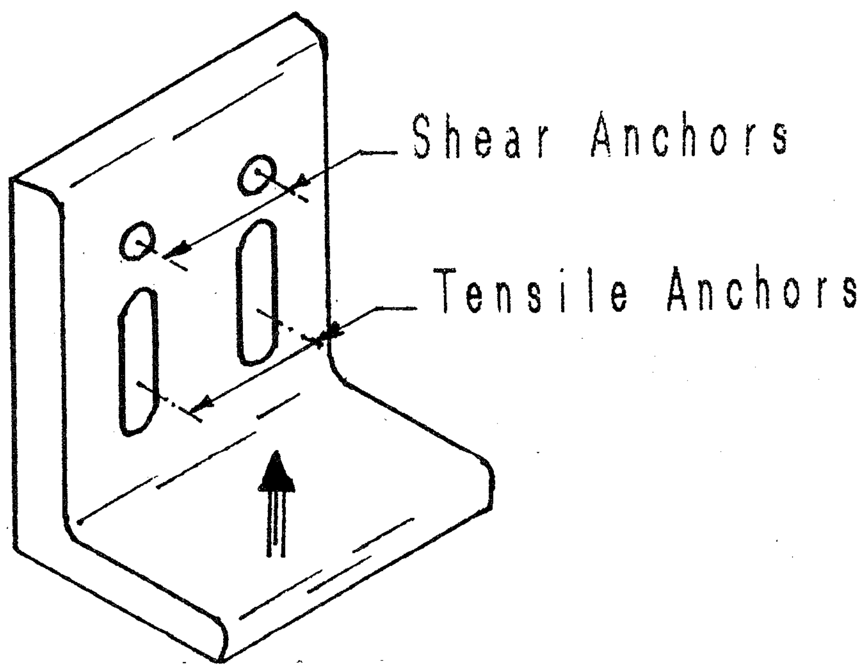


Figure 1.39. Controlling anchorage loads (from McCann [1991]).

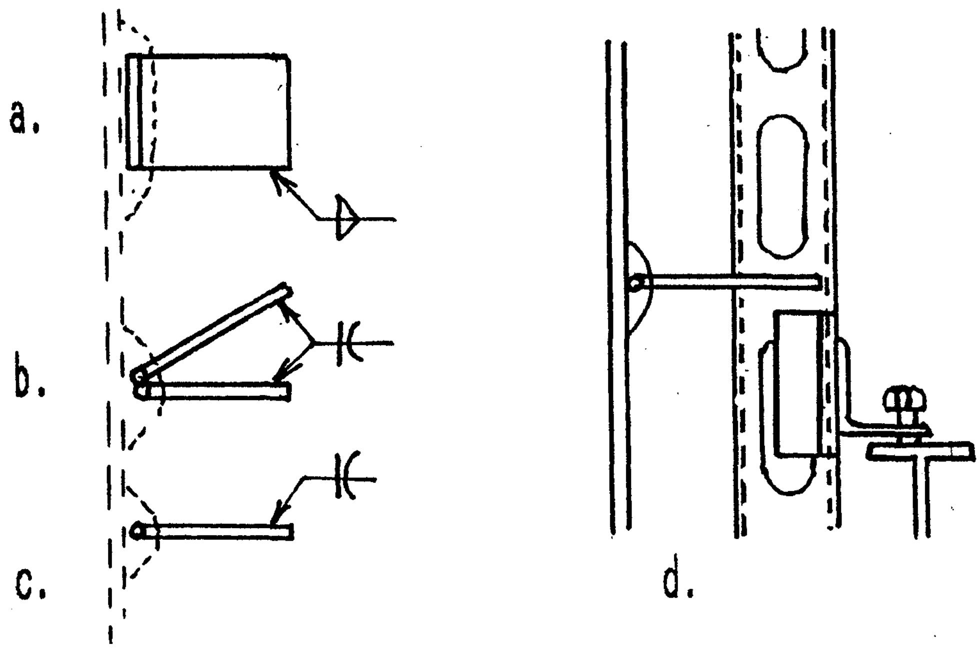


Figure 1.40. Glass fiber reinforced concrete (from McCann [1991]).

1.4.2 Cladding Panels and Connections: New Zealand

Hopkins, *et al.* [1985] presented information on architectural elements in earthquakes, a review of design and construction practice in New Zealand. In their abstract, the authors stated, "The paper describes the results of a survey of New Zealand and Californian designers, contractors, approving authorities and fabricators, conducted during 1984 as part of a research project for the U.S. National Science Foundation. The emphasis was placed on protection of architectural elements themselves, although inevitably the question of risk to people was addressed. The main sources of information were the response to a questionnaire sent to selected members of each affected sector and the material offered by those respondents who were interviewed. A clear picture of the New Zealand state-of-the-art emerged and a number of noteworthy examples of separation of architectural elements were identified. Recommendations for further research are made, particularly to improve knowledge of behavior, and of the economics of special protective measures. It is concluded that although New Zealand practice is advanced, there are important aspects which require attention."

The paper by Hopkins, *et al.,* includes sections on review of New Zealand practice, typical solutions, review of New Zealand Codes (NZS 4203:1976 General Structural Design and Design Loadings for Buildings), review of New Zealand literature and research, and design and construction practice in New Zealand. In the last section, there was a survey question on the design of application of precast concrete panels. The authors noted, "Precast concrete panels are widely used as the primary exterior claddings. Once the formal concept of the exterior building treatment has been resolved, the structural engineer tends to take over from the architect by designing the structure of the panels, erection methods, fixing methods and provision for movement. Precast manufacturers are often consulted on practical aspects, but do not generally carry out detailed design. Although there are some exceptions, which are cause for concern, precast concrete elements are competently handled by designers, manufacturers and constructors, in a wide variety of applications. In spite of the widespread use of precast elements there is a total lack of test of analytical data on the effectiveness of the solutions as built."

In their conclusions, Hopkins, *et al.*, stated, "In comparison with structural aspects, the subject of non-structural elements in earthquake has received scant attention in recent years. It is important that this imbalance be addressed as quickly as possible. The small amount of test results used to develop design criteria for New Zealand codes is an indication of this need and is a source of concern." They continued, "There is a need to clarify the drift provisions in codes of practice and to relate the performance in practice with a particular code provision. Code provisions in turn should related to expected actual displacements. Practitioners in New Zealand and California had varying understanding of the relationship between calculated and expected drift. This must surely be a point of basic concern." In addition, they noted, "There is a need for definitive testing on external wall elements, particularly glazing and precast panels. This needs to be done on a rational and scientific basis so as to relate it to code provisions. The results of racking tests which have been carried out need to be assembled and their relevance established in the overall context. It is disturbing to see the amount of potentially brittle and heavy material being placed on buildings where there is apparently little evidence of the performance of such materials in real earthquakes, or indeed in laboratory testing."

In their recommendations, Hopkins, *et al.,* offered suggestions for specific research topics. One suggestion is to "review available data on the performance of architectural elements in earthquakes in relation to current code provisions for drift limitations." Another suggestion is to "extend past investigations into response of yielding structures, to allow consideration of the effect on architectural elements. Such effects as damping provided, building period, height and mass, earthquake ground motions record and of course drift will be relevant."

In Appendix 2, Hopkins, *et al.,* included eight case studies, two of which are buildings clad with precast concrete cladding panels.

For the Wellington Teachers College, there is a description that includes references (from 1968 and 1970 from New Zealand journals), but no figures. "The complex comprises six buildings generally of two or three stories, with the common structural concept of reinforced concrete frames in both directions, supporting precast prestressed double T floors. Extensive use of prefabricated elements is made in forming the facade, including mullions, sun screens, exposed aggregate wall and spandrel panels... The structural framing system was seen as being relatively flexible. Consistent and thorough efforts were made to separate non-structural elements from the structural framing. Interstory movements of at least 12 mm were provided for... Most details are based on sliding brackets and/or resilient members. The details were revised with the clear objectives of minimizing hazard to people and of avoiding secondary damage at moderate levels of earthquake. In spite of modest height of these buildings, the separation of the non-structural elements was carried out as thoroughly and effectively as in any building in New Zealand. Because it was an early example and it has been well documented and described, concepts used have been used and improved in later buildings."

For the BNZ Centre in Wellington, there is a description, but no figures. The building is described as a "thirty story steel structure with floor comprising steel deck and concrete topping. Seismic resistance is provided by a perimeter frame with internal column for gravity support only. The external facade comprises precast concrete units supported outside the column line. Windows are built into the precast units which are detailed for interstory drifts. It is one of the largest buildings of its type in New Zealand... This building is a dominant feature in the Wellington central business district. Provision for earthquake movement in its facade elements was a prime consideration... Detailed analyses were carried out on the nature and magnitude of relative displacements of the precast concrete/window units and the structural frame, including tilting due to vertical movement of the supporting beams... The exterior precast concrete/window panels are designed to cater for 38 mm of interstory drift, which is three times the computed interstory drift under code loadings... Although completed in 1984, this building was designed in the late 1960s. It is therefore an early example of specific detailing for seismic movement in a major building."

Massey and Megget [1992] compiled a document on the architectural design for earthquakes, a guide to the design of non-structural elements. The document contains sections on measuring earthquakes, configuration, implications of NZ codes, structure and external walls, external wall types, windows and curtain walls, internal elements, partitions, suspended ceilings, and miscellaneous elements.

In the section on structure and external walls, cladding principles are outlined. "These principles primarily relate to external cladding of ductile structures. Four levels of participation of the cladding in the seismic resistance of the building can be identified:

1. "Theoretically complete detachment so that the cladding, usually lying outside the structure, does not contribute to its lateral stiffness at all: In practice, this would very rarely be the case. In a building with perhaps hundreds of cladding panels some transmission of forces from the structure to the cladding, and vice versa is likely, even if the cladding is comparatively lightweight, but this may not be significant in the overall structural response.
2. "Accidental participation of cladding in the seismic response: This can occur during an earthquake due to the separation distance being too small (if the cladding lies within the structural frame), or binding of supposedly free-moving connections of cladding to structure.
3. "Controlled stiffening or dampening of the structure by the cladding and its attachments: So far this approach has rarely been used, but it is the subject of research, especially in the United States. It could be a useful future development.

4. "Full integration of the cladding into the structural system: Where the cladding and the structure are homogeneous, as for instance when in situ concrete walls are used, the results are predictable and the cladding becomes part of a shear wall system. It is also technically possible to achieve integration using precast components, at least in low-rise situations. But in practice this approach has not been widely employed. It is seldom that the architectural cladding design is fully compatible with the structural concept: If it is not, then configuration problems can arise.

"In practice, therefore, detachment of cladding has remained by far the most common approach, despite its inefficiency - in the sense the more economical and seismically efficient structures could be produced by integration."

In the section on external wall types, "facing materials" are defined as those "which clad the main structure but should be effectively detached from it for seismic design purposes... Facing panels are commonly large units, often covering the full width of a structural bay for a story height. Construction handling is a major determinant in sizing such panels - either because weight must be limited (usually for ease of cranage), or because over large panels become impractical to handle."

The authors continued, "When large heavy facing panels are used, provisions for seismic movement becomes critical. This is usually achieved by having fixed bearing connections at the top of the panels and providing for lateral movement in detailing the bottom fixings. But these locations are sometimes reversed. The fire rating of fixings can be an important consideration."

"Two common types of connections have been developed to allow for movement between heavy panels, such as precast concrete, and the main structure: (1) rod connections, which are the most common type in West Coast, USA; and (2) sliding connections, which have been widely used in New Zealand and elsewhere. Neither approach is without disadvantage and, in each case, predicted performance is based on engineering principles, rather than observed performance after an earthquake, or even extensive laboratory testing."

"Rod connections are commonly referred to in the USA as 'push-pull' connections. The rod and connector details must be tough enough to withstand imposed loads, both on the face of the panel and 'in-plane' yet the rod must be long and flexible enough that it will remain ductile over the predicted range of movement. If the panels are to be fixed close to the structure, this may be difficult to achieve." Figures given by Massey and Megget were taken from Arnold, *et al.* [1987]. "'Push-pull' connection seem to have performed well, but some queries about their long-term effectiveness have been raised. The connection of the rod to the panel is particularly critical." The authors referred to Rihal [1989].

"Sliding connections are usually provided by using cleats to join the panel to the structure. Each cleat is bolted through a slotted hole to provide for movement. Disadvantages of this system are [as follows]: (1) movement will not occur if the clears 'bind' due to misalignment during construction or flexure of the components under load, or 'seizing' of the detail over time (due to rust); and (2) bolts may be overtightened so that lateral loads are transferred through the connection. Sliding connections are best kept to situations in which the degree of lateral movement in each connection in small, e.g., stiffer structures, or panels of reduced height."

The authors noted that "movement can occur at right angles to the panel face (or at any other angle, depending on the direction of the drift in the structure). Connections must allow for this movement... Finally, the designer must be aware of the relative movement of panels at corners, both internal and external, so that drift does not lead to impact between panels at these locations."

"Of the many different ways of arranging panels four have been chosen as fairly typical." These include:

1. "Story height panels, which may be continuously solid, or incorporate 'hole-in-the-wall' windows. There has been a return to this type of approach in recent years as architectural trends have changed.
2 "Spandrel panels, often approximately half story height, from window head to the sill of the next story, but can be no more than a beam facing where more glass is used.
3. "Complete facing of columns and beams using separate panels for each purpose, with movement joint between each panel. Note that this approach may reduce the amount of movement to be accommodated at each joint.
4. "Other approaches are L or T-shaped panels, or even double-story height arrangements, if heavy cranage is available."

These panel arrangements are very similar to those used in the U.S. (See figures included in the section on U.S. cladding design practice.)

Charleson [1992] prepared a document on guidelines for the use of *structural* precast concrete in buildings. This reference does not cover precast concrete cladding panels, but it is mentioned here for the interested reader. As noted in the introduction, "...only structural elements are dealt with since architectural (non-structural) precast concrete is not normally designed to contribute to the overall structural integrity and requires a different set of design criteria. Although the focus is on seismic aspects, many sections refer to gravity load effects as well as volume changes such as creep, shrinkage and thermal actions, since these effects can result in a significant reduction in seismic performance."

1.4.3 Cladding Panels and Connections: Japan

Yashiro and Sakamoto [1991] presented information on tall building cladding systems in Japan case study on several innovative examples during 40 Years after World War II. The authors noted that "Japan is one of the countries where industrialized production systems of buildings are well developed. Among building elements prefabricated in factories, certain proportion of so called curtain wall components and external wall panels are produced corresponding to individual requirements of each project, while most of the others are produced corresponding to general requirements in the market. In this sense, curtain wall components and external wall panels are particular-demand-oriented components while the others are general-demand-components. In Japan, independent professions such as architects and building engineers have been deeply concerned in technology development and innovation of cladding components in case of particular-demand-oriented components more than in the case of general-demand-oriented components. This paper deals with trial and error process of Japanese architects and engineers in the fabrication and detailing of tall building cladding systems since the 1950s until the middle 1980s. These processes suggest the importance of reconsideration of regional natural and social characteristics in the introduction of internationalized building technology."

Yashiro and Sakamoto continued, "This paper introduces examples of unique trials in fabrication and detailing of tall building cladding design practice since 1945 until 1985 in Japan. In the first chapter, how Japanese architects and engineers learned the design practice of curtain walls [is] reviewed briefly. In the second chapter, the contribution of Japanese craftsmanship to innovative fabrication of tall building cladding components is illustrated with several examples including examples of cladding finishes. In the last chapter, Japanese severe natural conditions are explained first, and detailing for earthquakes and rainfalls are exemplified. Through the consideration here, factors of craftsmanship and mindfulness for several natural conditions affect [an] important role in innovation of fabrication and detailing of tall building cladding system in Japan during four decades after the second world war."

Yashiro and Sakamoto discussed innovative detailing for earthquakes in their Section 3.2 They presented principles for accommodation of story drift, including the sliding and rocking mechanisms. For the sliding mechanism, "the joints at the upper or lower floors are the roller type in order to allow relative displacement." The top two corner connections are rollers or long horizontal holes, and the bottom corner connections are noted as pins and gravity supports. For the rocking mechanism, "the joints are design so as to allow the story drift." The top and bottom corner connections are rollers or long vertical holes, and the bottom corner connections are noted as gravity supports, as shown in figure 1.41 (taken from fig. 23, a two-page figure in the paper).

Sakamoto and Yashiro [1992] presented information on innovations and cladding materials and systems in Japan. They stated, "By the 1960s precast concrete curtain-wall technology borrowed from other countries was common practice, but it was not until the 1970s that many new techniques for improved curtain-wall design were actually developed in Japan. A 'rocking mechanism' designed by Nikken Sekkei and described (by Wang [1992]) was developed to accommodate the story drift affecting the large height-to-width ratio of the precast concrete panels used in the IBM headquarters building in Tokyo." These panels are typically tall, narrow column covers and wide, short spandrel beam covers. "Figure 1.42 (taken from fig. 6.3 in the paper) shows the potential relative displacement of the cladding panels, while figure 1.43 (taken from fig. 6.4 in the paper) illustrates the actual attachment mechanism."

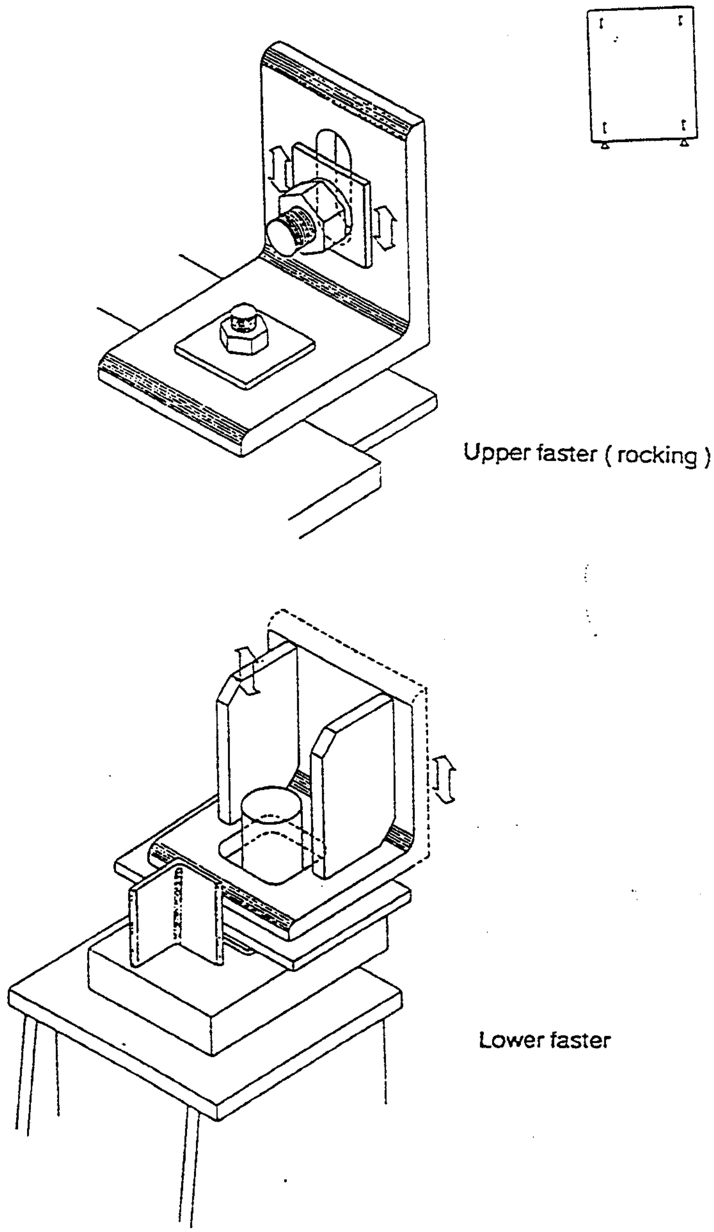


Figure 1.41. Details for story drift in practice, part 1 of 2 (from Yashiro and Sakamoto [1991]).

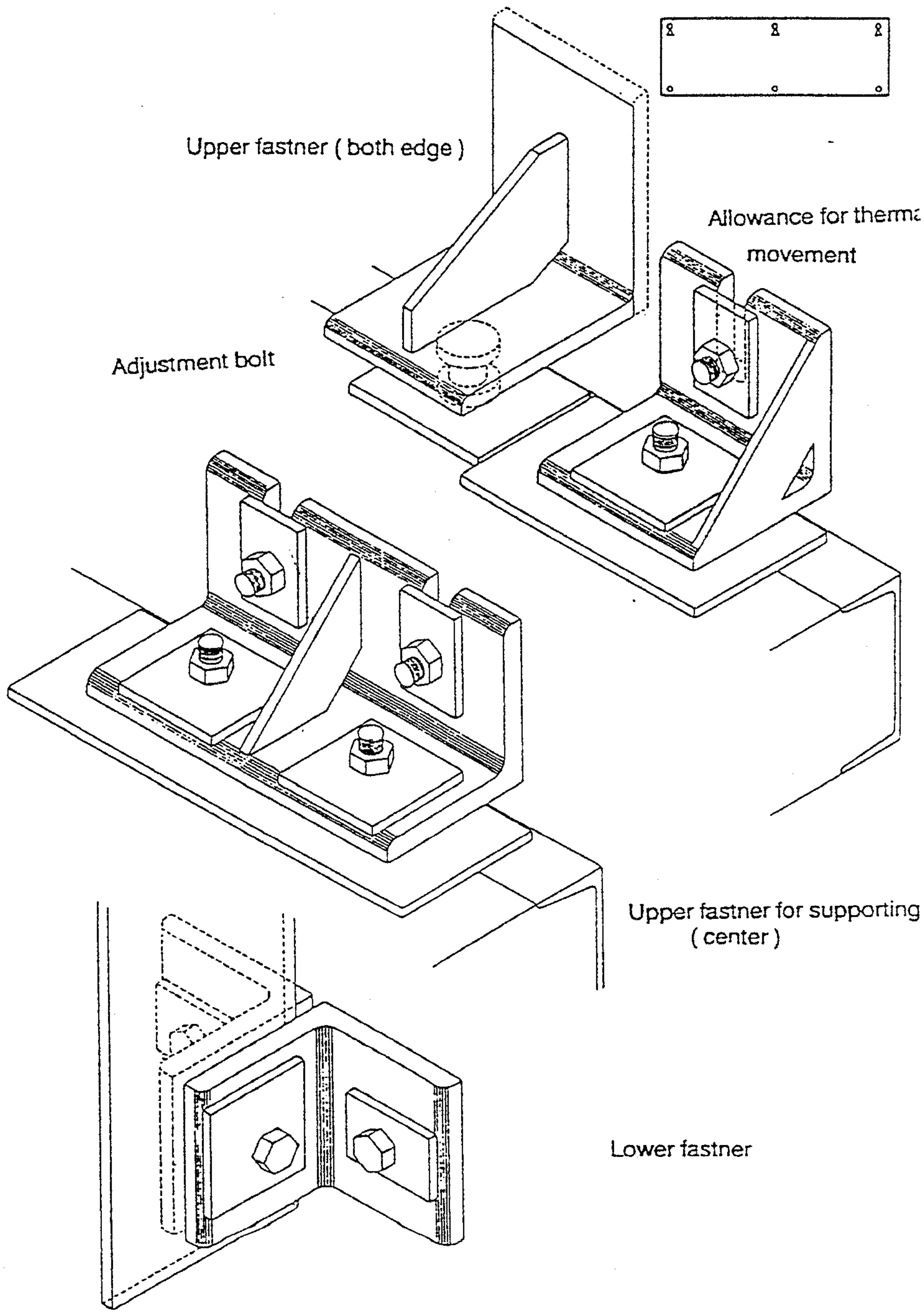


Figure 1.41. Details for story drift in practice, part 2 of 2 (from Yashiro and Sakamoto [1991]).

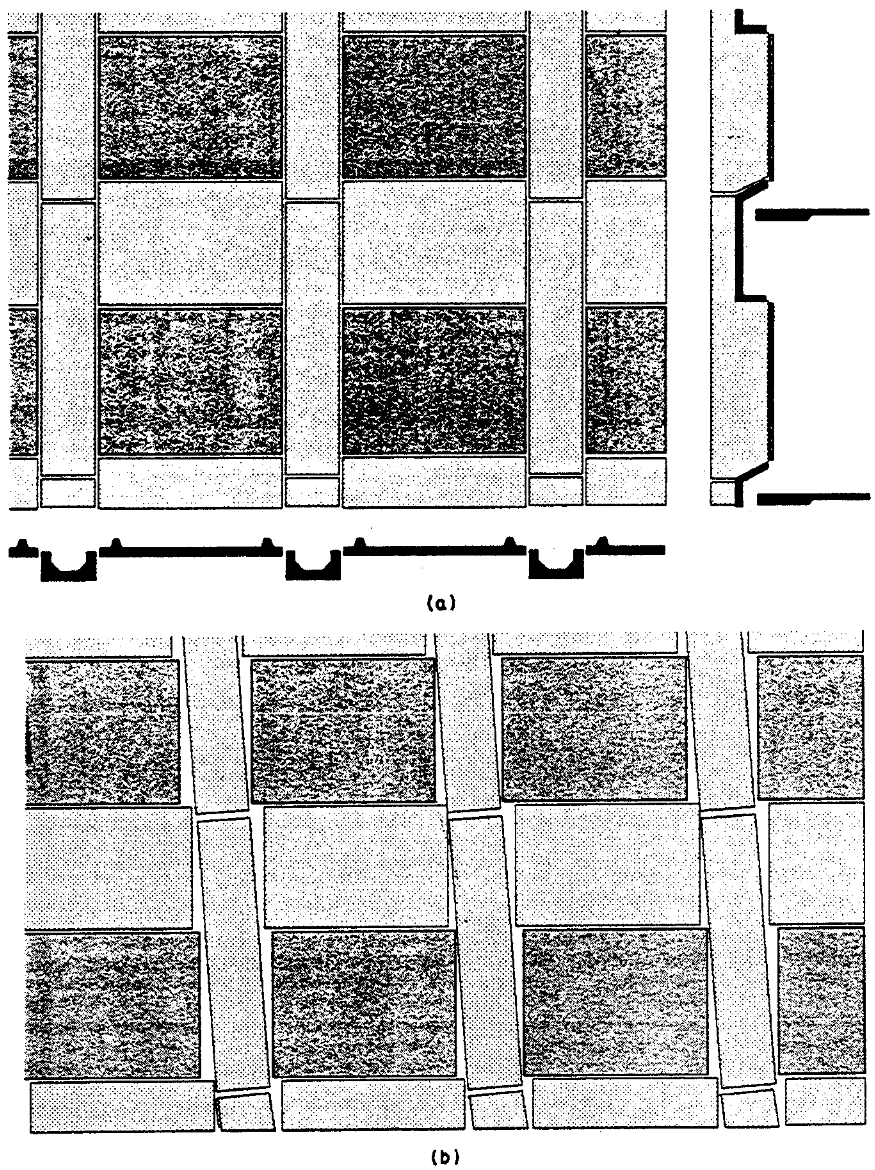
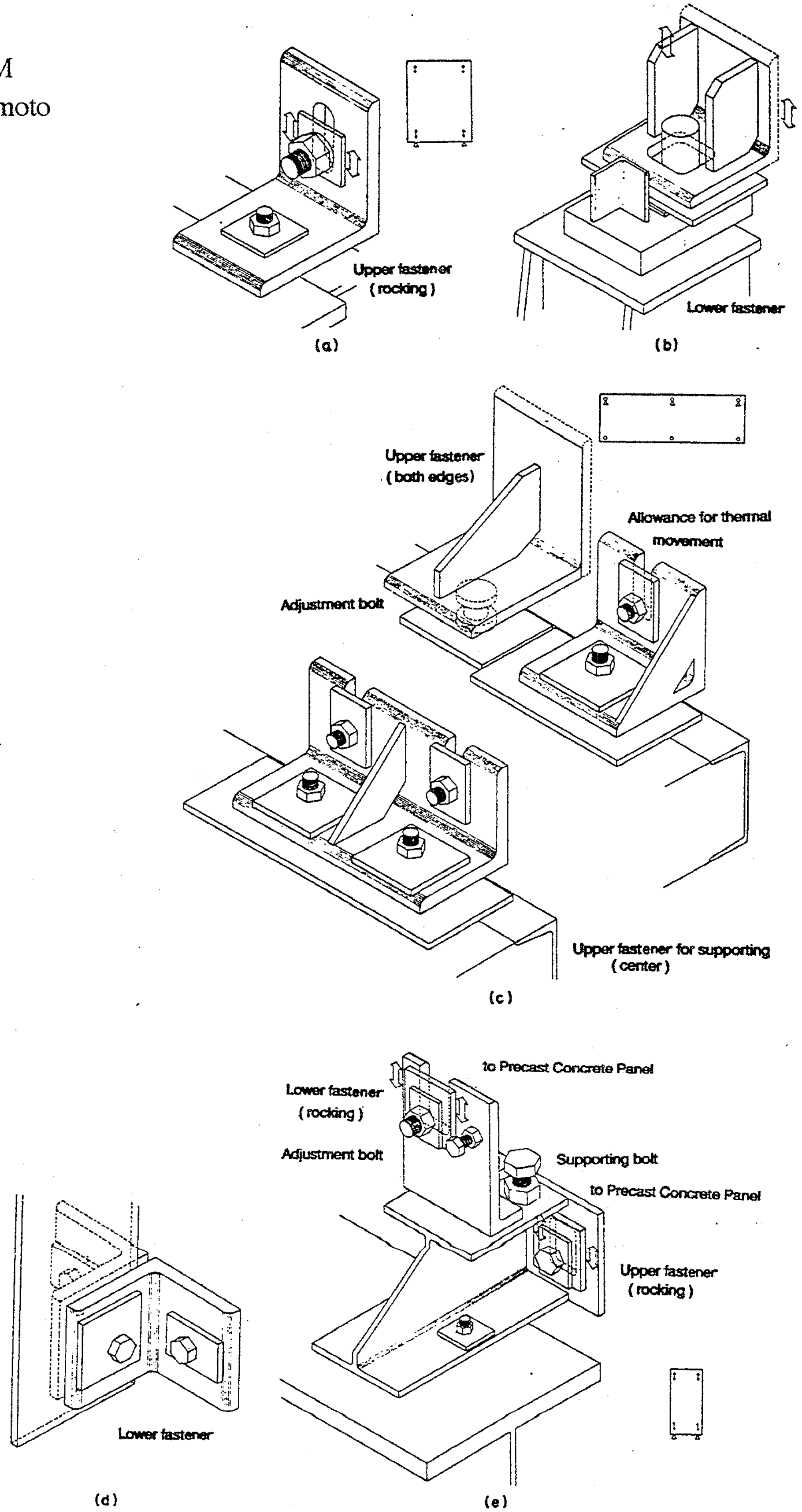


Figure 1.42. Potential displacement of cladding system on Tokyo IBM headquarters (from Sakamoto and Yashiro [1992].

Figure 1.43. Rocking mechanism in Tokyo IBM headquarters (from Sakamoto and Yashiro [1992].



1.4.4 Cladding Panels and Connections: Canada

Smith and Gaiotti [1989] and Gaiotti and Smith [1992] conducted an analytical study on the interaction of precast concrete with a story-height frame module and on the stiffening of moment-resisting frames by precast concrete cladding. Earthquake-resistant design is not explicitly addressed.

The abstract from the authors' first paper is as follows: "Precast concrete cladding panels are usually assumed as non-structural in function, and sliding connections are introduced in the plane of the panels in attempting to avoid interaction with the structural frame. However, comparisons of field tests on buildings have suggested many times that non-loadbearing cladding panels, connected in their usually recommended ways, contribute significantly to the lateral stiffness of the structure. The purpose of this paper is to describe a study of the interaction between a typically connected precast concrete cladding panels and its supporting structural frame. The significant parameters and their relative importance in influencing the structure's racking stiffness are described. It is shown that the 'forward' double-curvature bending of the supporting beam, associated with the racking of the frame, and the 'backward' double-curvature bending related to me rotation of the panel counteract each other to cause the stiffening of the composite module. It is concluded that, in using the types of panel-to-frame connection arrangements recommended by the CPCI and the PCI, cladding panels will significantly increase in the in-plane lateral stiffness of the structural frame. An analogous spring model has been developed to better visualize the actions involved when the frame with the panel and its connections are subjected to a horizontal load."

From Smith and Gaiotti [1989], figure 1.44 (taken from fig. 3 in the paper), "the panel is connected to the frame by two bearing connections, 1 and 5, near the bottom of the panel, and four tie-back connections, 2, 3, 4 and 6. Details of typical loadbearing and tie-back connections are shown in the diagrams of connections 1 and 2, respectively, in figure 1.45 (taken from fig. 4 in the paper). Bearing connection 1 also constrains lateral displacement of the panel in its own plane, while connection 5 differs in allowing lateral movement, by having neoprene pads on each side of the HSS section. In connections 2, 3 and 4, which are identical, vertical movement is allowed by the oversize hole in the angle, with the vertical slot in the attached plate; however, in-plane lateral displacements are restrained. The elongated hole in the angle-leg welded to the slab permits adjustment during erection. The angle in connection 6 is the same as in 2, 3, and 4, except that the plate has a horizontal slot to permit in-plane lateral motion.

"The connections described are typical of those used in Montreal and other eastern cities, and conform in the design, location and restrain conditions with the recommendations in the design manuals."

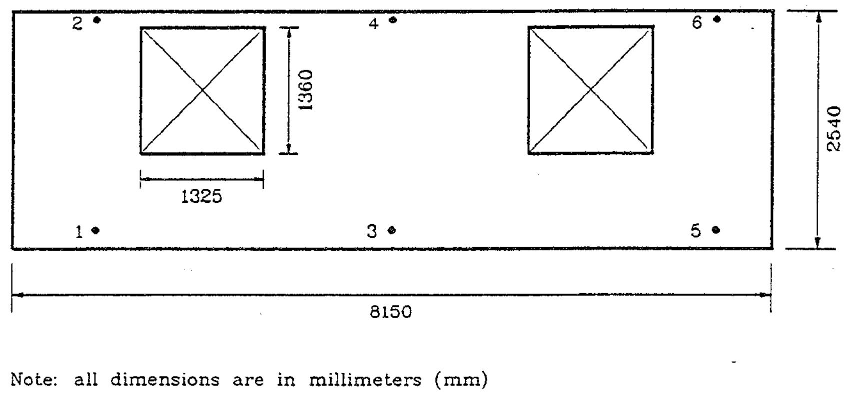


Figure 1.44. Location of connections in representative panel (from Smith and Gaiotti [1989]).

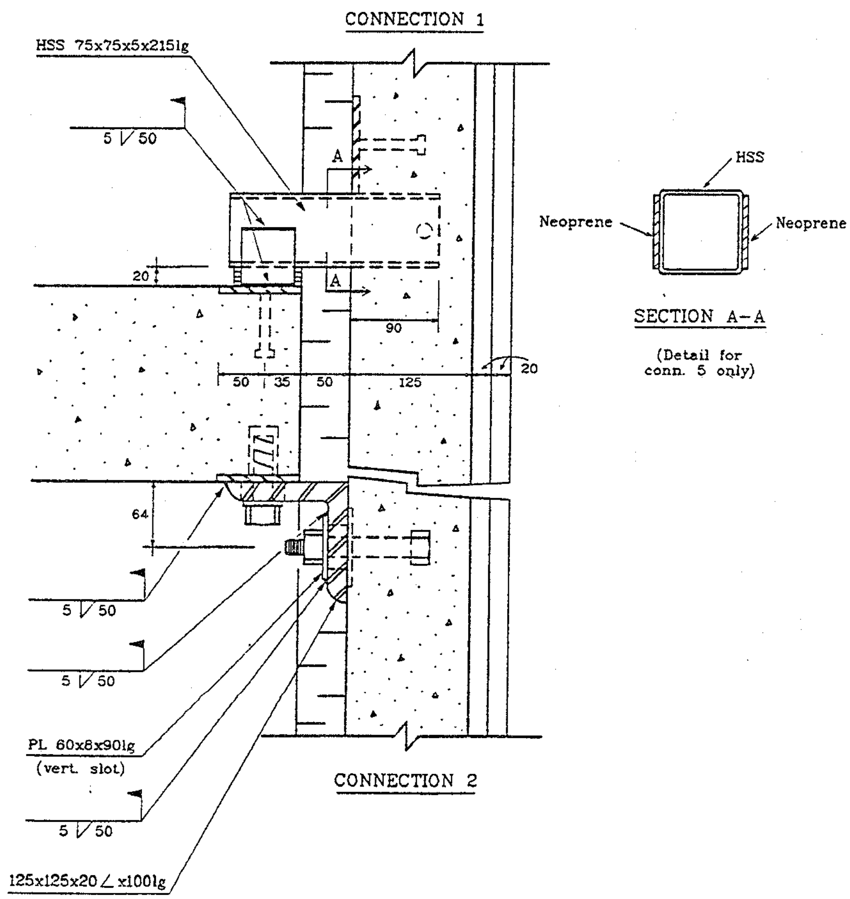


Figure 1.45. Details of typical connections (from Smith and Gaiotti [1989]).

1.4.5 Cladding Panels and Connections: Joint U.S.-Japan and U.S. Studies

In the middle 1980s, a U.S-Japan Cooperative Research Program included experimental studies on commonly used cladding connections in the U.S. and Japan, and in the U.S. The U.S.-Japan research projects of Wang [1986a, 1986b, 1987, 1992], and Foutch, *et al.* [1986], and the U.S. research project of Rihal [1988a, 1988b, 1989] are outlined in this section.

RESEARCH GROUP: The Dept. of Architecture, Univ. of California at Berkeley, and the Building Research Institute in Tsukuba, Japan.

References: • Wang [1986a]. *Nonstructural Element Test Phase: U.S.-Japan Cooperative Research Project on a Full Scale Steel Test Frame.* • Wang [1986b]. "Full Scale Tests of Cladding Components." • Wang [1987]. "Cladding Performance on a Full Scale Test Frame." • Wang [1992]. "Design of Cladding for Earthquakes."

Note: Wang [1987] is used for the basis for this section.

Type of Study: Experimental.

Abstract: "The last phase of the recent U.S.-Japan Cooperative Research Program's full scale steel structure tests concerns the seismic performance of so called 'nonstructural' or 'extrinsic' elements. Both Japanese and U.S. elements were installed onto the full scale, moment resistant frame; static test of the frame with cladding and internal elements took during during three weeks of July 1984 at the Building Research Institute in Tsukuba, Japan. The U.S. side test focused on (precast concrete and glass fiber reinforced concrete) cladding, and the Japanese side oversaw testing of cladding common to Japanese practice, and internal partitions and ceilings common to both U.S. and Japanese practice. This paper describes findings of U.S. cladding performance tests with regard to values of seismic story drift designated in the *Uniform Building Code,* and observations on the behavior of Japanese elements."

Cladding Panel and Connections, and Comments by Wang: "Two types of mechanisms which enable cladding panels to accommodate drift described in" figure 1.46 (taken from fig. 8 in the paper). The rocking mechanism is comprised of connections that are designed with slots or oversize holes to allow rocking motion as shown, and to accommodate story drift. The lower connections are bearing, however, should they fail, the upper ones can also support panel dead load. The swaying mechanism is comprised of top connections that accommodate in-panel story drift, by the use of slotted holes or flexible, long rods. The lower connections are relatively fixed and are bearing and should be somewhat ductile. "For each case, the objective is to avoid stresses which would lead to brittle panel or connection failures. In the U.S. the 'sway' or 'translation' mechanism is common, although tall column covers are sometimes design to rock; in Japan, the 'rocking' mechanism prevails. Both of these mechanisms isolate the cladding elements from the steel frame in order to minimize interaction between panels and structure."

"While the location of bearing connections at the bottom of wall panels is common in U.S. practice, the Japanese engineers were aghast at this arrangement. If the lateral connections completely fail, there is nothing left to resist the panel's tendency to rotate outward, subjecting the bearing connections to an overwhelming moment. In general, the Japanese engineers were dubious and incredulous of the fact that the details on the U.S. test side were even remotely representative of American practice, since the connection looked so simple and vulnerable to disastrous failure mechanisms. Even after the tests showed that several of the U.S. connections had excellent behavior, the Japanese engineers continued to doubt the reliability of the American connections in a major earthquake."

"The Japanese researchers' confidence of excellent performance from rocking connections was confirmed in the test. Except for a connection with a mis-installation, all Japanese panels and connections successfully survived the entire loading sequence. A conceptual advantage of this detail lies in the vertical alignment of the slots and the 'rocking' displacement of the panels. The result is that the sliding components of the connections do not need to accommodate large distances relative to the actual story drift of the frame, and the distance which needs to be accommodated by the connection is a function of the horizontal distance between connections at a level, not vertical distance. For tall panels, particularly column covers exceeding one story, this aspect of rocking motion is especially desirable, and the Japanese initially developed the rocking concept to deal with tall panels."

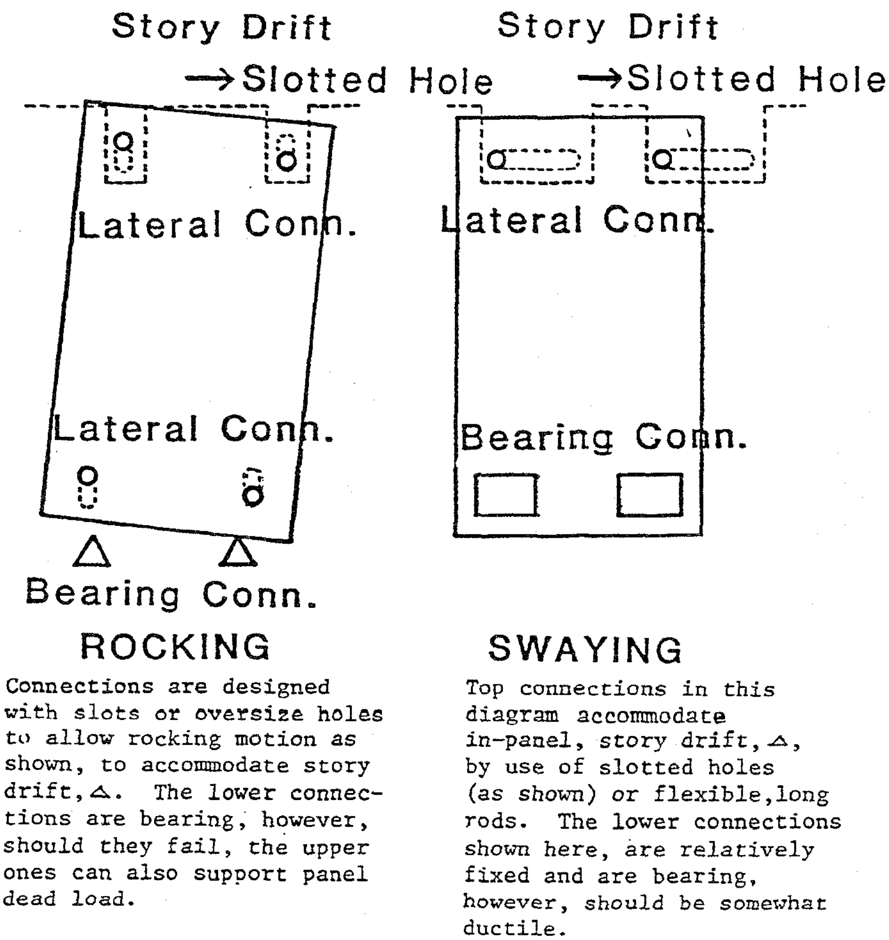


Figure 1.46. Mechanisms of drift and accommodation in cladding design (from Wang [1987].

Wang continued, "Rocking details such as the Japanese connection (see fig. 1.47 taken from fig. 9b in the paper) are not common in the U.S., although they are more frequently used in projects with tall panels. Resistance to wide adoption of rocking connections in this country will probably continues for two main reasons: (1) the detail is more expensive; and (2) though it is straightforward in principle, is requires far more complex details than U.S. connections. Nearly every U.S. structural engineer who was shown the construction drawing of the Japanese connection felt that the detail's complexity lead to a greater chance of errors and improper installation. One San Francisco engineer described the Japanese detail in the full scale test as a 'Swiss watch,' adding 'I doubt the net effectiveness (of it).' Another Bay Area engineer conceded that the rocking connection accommodates drift the best, however its assembly 'requires a lot more hardware (than U.S. connections), is much too complicated, and requires tighter field controls for panel erection' with the result being much more expense than for the sway type connection. (The mis-installed rocking connection substantiates this concern, especially when one considers that the test specimen had exceedingly careful workmanship.)"

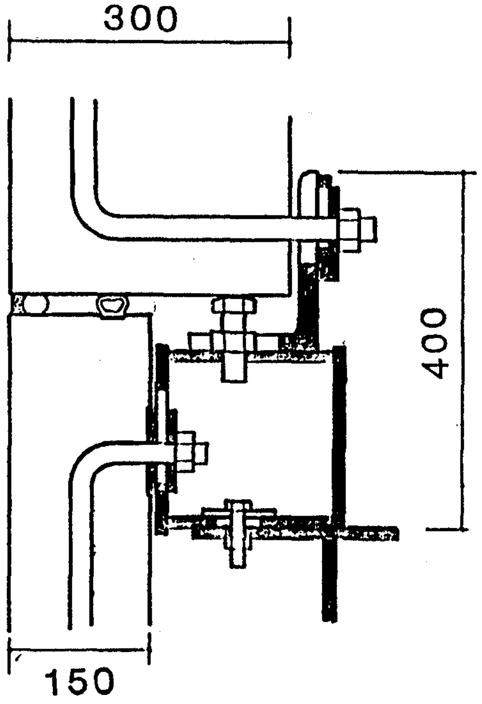


Figure 1.47. Schematic diagram of Japanese rocking connection on precast concrete panel (from Wang [1987]).

Experimental Program:

Objectives: "The overall objective of this phase of the U.S.-Japan project is to investigate seismic issues of extrinsic element performance with which structural and architectural designers are most concerned. The test method allows us to clarify the relationship between story drift and damage of extrinsic elements design to satisfy Japanese or American code requirements. The cooperative nature of the project provided several interesting performance comparisons between respective design practices in the U.S. and Japan.

"The designs of the U.S. elements in this project do not necessarily represent the best details used in American practice. Since the goal is to demonstrate the performance of commonly used details, less than ideal design practice is included as long as it conforms to the minimum code requirements. Cladding design drastically differs from region to region in the United States; the cladding in the test is representative of Northern California practice.

"Several issues of configuration and engineering design can lead to details normally considered acceptable, but are in fact not desirable. The project confronts the possibility that details which are now regarded as normal and acceptable, should perhaps be reconsidered in light of how they really respond to large seismic drifts. Joint size limitations, corner cladding configurations, and lateral connection arrangements are some examples where architectural, installation, and aseismic requirements may clash. This project investigates commonly encountered problems in the design of cladding, and assesses which currently acceptable details are in fact inadequate."

Description of Test Frame and Specimens: "Typical floor height for the six story structural steel frame was about 3.40 meters (11 feet); each side of the square plan had two bays of 7.5 meters (25 feet). The three-dimensional test specimen demonstrated behavior and interaction of cladding which isolated assemblages would not." A 3-D diagram and plan of the steel test frame are shown in figures 1.48 and 1.49 (taken from figs.1 and 2 in the paper).

Figure 1.50 (taken from fig. 4 in the paper) shows the three elevations of the frame onto which American and Japanese cladding elements were installed. No explanation is given why two story-height panels were used side-by-side between columns. (The use of two panels carries over in the Georgia Tech work described later on.) As noted in PCI [1989], this is not a common configuration for precast concrete cladding panels.

"Precast concrete and glass fiber reinforced concrete (GFRC) panels were tested with a variety of sway type connections that are commonly used in the western Unites States."

Type of Loading: "The Japanese side conducted free vibration and forced vibration tests before and after installation of extrinsic elements to ascertain the stiffness and period of the structure." For the clad frame, quasi-static loading with increasing displacement amplitudes were used. "The static loading sequence culminated in a $^1/40$ story drift ratio (0.025h, where h = story height) which nearly reached the jacks' capacity. This level closely corresponds to both a credible drift in a major earthquake and to *UBC* design drift requirements... Loading jacks applied one direction of horizontal displacements on the frame which resulted in approximately the same story drift at each level."

Instrumentation: The interested reader is referred to the paper. Strain gauge readings were plotted as stress versus load step number.

Observations: The interested reader is referred to the paper.

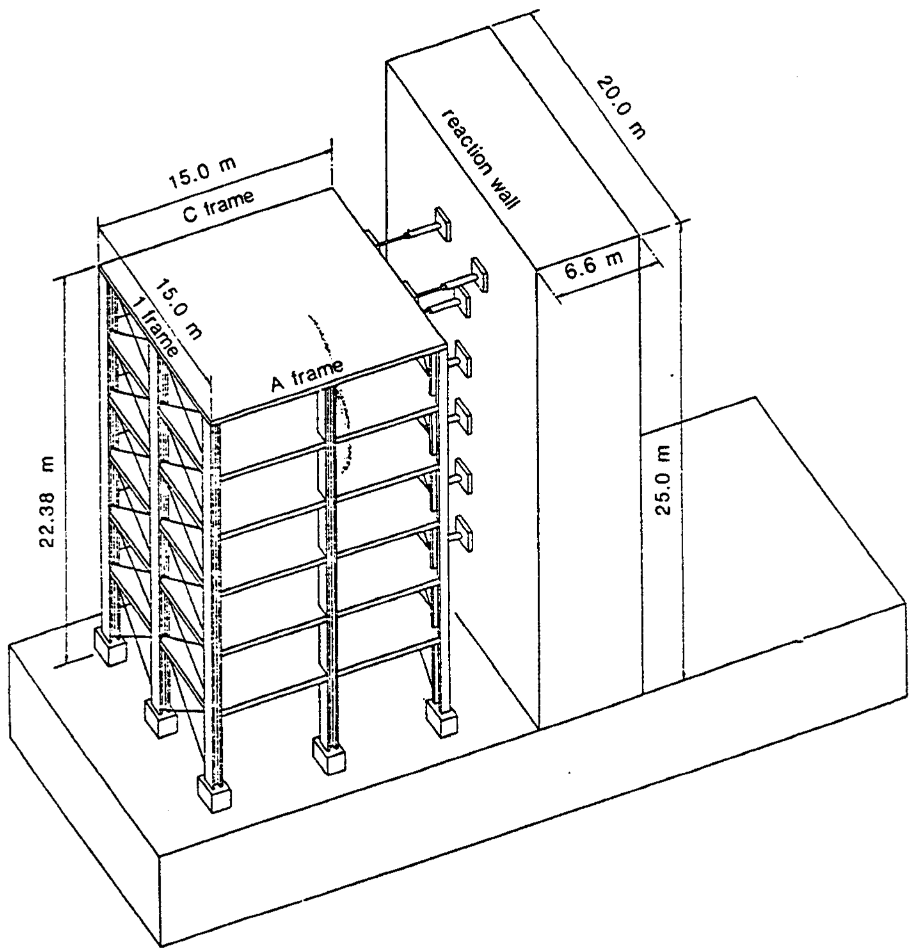
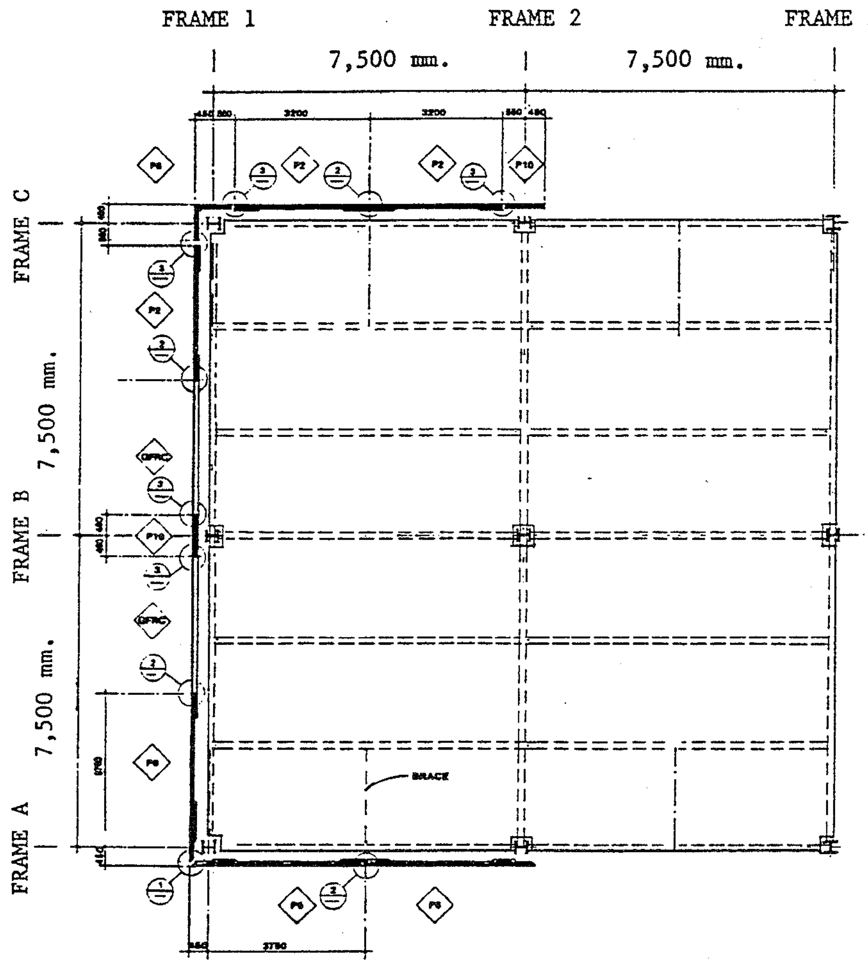


Figure 1.48. Full scale six story steel frame (from Wang [1987]).

Figure 1.49. Plan of clad frame at floor 2 (from Wang [1987]).



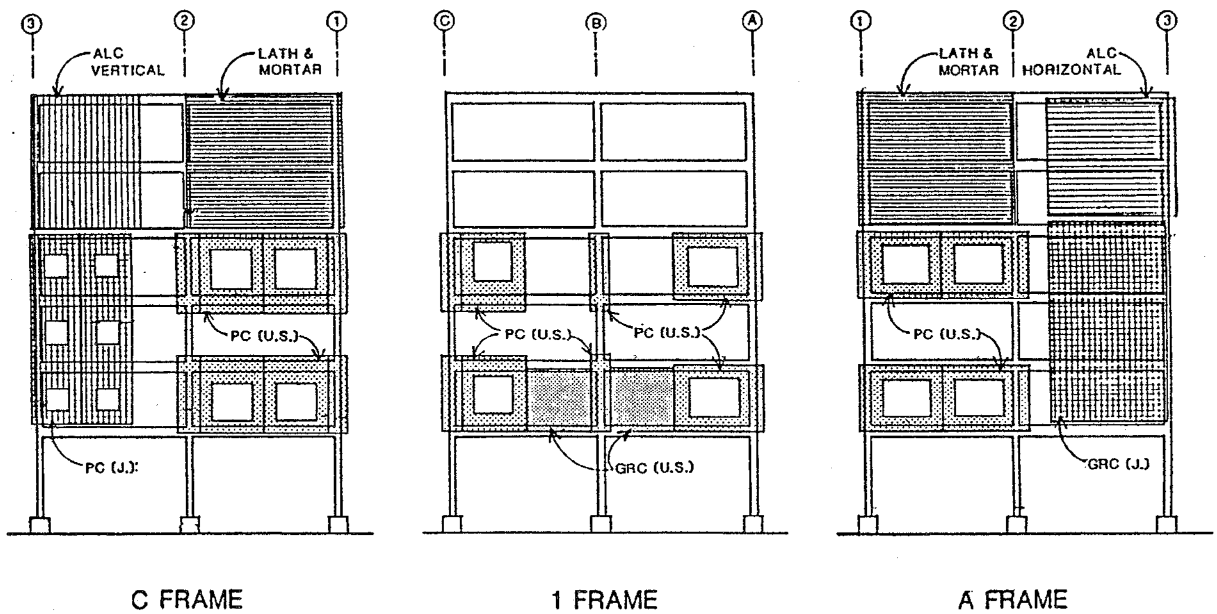


Figure 1.50. Elevation of frames with U.S. and Japanese cladding elements (from Wang [1987]).

Summary of Design Implications and Recommendations: "The survey of Bay Area practitioners revealed a large range of opinion on many issues dealing with the performance and design of cladding subject to seismic drift. The U.S.-Japan tests focused on several of these issues, and produced data which may affect design principles for cladding.

1. "Long ductile rods used for lateral connections can accommodate very large story drift, but sliding connections may have problems either due to insufficient slot length or impedance of the sliding mechanism. Although it is possible to design a sliding connection that enhances their reliability, they are still potentially fraught with problems ranging from weathering and aging of the connection, to improper installation, or poor detailing. Lateral connections, in particular, should not depend upon subjective criteria for installation such as tightening of nuts which cannot be easily perceived during inspections. Once the connection's sliding mechanism is impeded, the failure of the connection may be sudden and dangerous. If sliding connections are to be allowed, they must be detailed such that correct installation does not require great experience and skill on the part of the installer. Slot length needs to be generous, to avoid imposition of large stresses in panels and connections.
2. "Bearing connections must be sufficiently flexible to avoid conveying stress to the panel, resulting from interstory drift in regard of both in-plane and out-of-plane components of direction. The choice of tube or angle connections makes a great difference in the degree of cracking of the panels. Care should be taken to not inadvertently stiffen connections, such as pouring new concrete around the connection body.
3. "Panels should be 'hung' such that bearing connections are at the top and lateral connections are at the bottom, whenever possible. The common practice of bottom bearing connections may result in falling out of panels if the lateral connections fail.
4. "Connections from a panel to frame should be oriented in the same horizontal direction, or else extensive warping and cracking of the panel will occur. This caution is particularly noteworthy in design of cladding for corner conditions.
5. "Joints must be wide enough to avoid contact between panels as a result of drift. Adjacent panels should be designed to respond to drift, in a similar manner whenever possible. Thus, placing wall panels attached to girders, next to column covers attached to columns, must be detailed with extreme caution, to avoid 'bumping' (pounding) of adjacent panels."

RESEARCH GROUP: University at Illinois at Urbana-Champaign, and the Building Research Institute in Tsukuba, Japan.

Note: This reference is included, because it offers additional insight into the tests performed by Wang [1986a, 1986b, 1987, 1992], and about the influence of alleged nonstructural elements, in addition to the heavy cladding panels.

Reference: Foutch, Goel, and Roeder [1986].

Type of Study: Experimental.

Abstract: See Wang [1986a, 1986b, 1987, 1992] for description of Phase IV.

Experimental Program:

Objectives: See Wang [1986a, 1986b, 1987, 1992].

Description of Test Specimens: See Wang [1986a, 1986b, 1987, 1992], and description and figures (see pages 54-55 and 57-59 in this document, under the "Research Group" from the Department of Architecture at the University of California at Berkeley, and the Building Research Institute in Tsukuba, Japan).

Type of Loading: "The Phase IV testing did not use seismic acceleration simulation as employed in the first three phases. Instead, each floor was subjected to a cyclic story drift (of quasi-static story drifts)... It must be noted that there are severe limitations with this test method. It does not consider the mass and velocity and acceleration of the nonstructural panel, since a true dynamic test is required to include these dynamic components of the response. However, the test does provide a reasonable indication of the behavior of the elements under large story drifts, and the effects of the elements on the strength and stiffness of the structure."

Tentative Observations: "...Joint slip was first observed at story drifts in the order of $^1/700$. Initiation of cracking in joint sealants was first noted at story drifts in the order of $^1/500$. Damage to the nonstructural elements increase dramatically with increasing story drift, and it was sensitive to the type of installation detail and errors in installation. The construction personnel appeared to be very conscientious by U.S. standards, but a number of errors in the installation of nonstructural elements were noted. Several premature failures could be attributed to these errors in installation. The long ductile rod attachment detail generally performed much better than the short bolt-slotted hole concept. It permitted larger movements and transferred smaller forces than the slot hole element. As a result, nonstructural elements generally suffered less damage with these attachments. The corner elements appeared to be a source of major problem, and more study is needed in this area.

"Ceiling tile elements suffered no damage until the story drifts reached $^1/150$, and the damage was significant only after the story drift exceeded $^1/125$. Several attachment details were regarded as being in a dangerous condition after the story drift exceeded $^1/60$. Two types of door and door jamb assemblies were tested. Both were built by Japanese manufacturers, but one was design for seismic applications in that it was designed to accommodate larger movements. The ordinary doors became impossible to operate at story drifts greater than approximately $^1/500$, and the seismic designed doors were impossible to open at displacements greater than $^1/125$.

"Finally, it should be noted that the nonstructural elements had considerable impact on the structural properties. ...nonstructural elements reduced the natural period by 30%, and this would suggest that the overall structural stiffness was increased by more than 100%. The stiffness decreased with damage to these elements. After 8 cycles (maximum story drift $^1/350$) however, most of this additional stiffness had been lost."

RESEARCH GROUP: California Polytechnic State Univ., San Luis Obispo, CA.

References: • Rihal [1988a]. "Seismic Behavior and Design of Precast Facades/ Cladding and Connections in Low/Medium-Rise Buildings." • Rihal [1988b]. "Earthquake Resistance and Behavior of Heavy Facades/Claddings and Connections in Medium-Rise Steel-Framed Buildings." • Rihal [1989]. "Earthquake Resistance and Behavior of Architectural Precast Cladding and Connections."

Note: Rihal (1988a] is used for the basis for this section.

Type of Study: Experimental and Analytical.

Abstract/Summary: "Seismic behavior and design of heavy facades/claddings and connections in buildings has been investigated, and unique cyclic racking tests of representative precast concrete facade/cladding panels and connections have been carried out. During the first major phase of the research project current practices for design and detailing of heavy facade/claddings and their connections to supporting structural systems, were evaluated. In consultation with practicing architects, engineers, researchers, and facade/cladding manufacturers, state-of-the-art data for facade/cladding design, detailing and erection practices were compiled. Available data on the performance of building facade/cladding during previous destructive earthquakes including the recent Mexico City Earthquake of September 1985 was evaluated. Analytical and experimental techniques of modeling the seismic behavior of heavy precast concrete facade/cladding panels and connections have been investigated. The role of modern testing methodology in assessing the seismic behavior of building facades/claddings and connections has been evaluated. Pilot static tests of typical ductile (push-pull) cladding connections were carried out to investigate the strength and behavior of these connections. Cyclic in-plane racking tests of a full-size precast concrete cladding panel with bearing connections at the bottom and ductile (push-pull) connections at the top, representative of California current practices, has been carried out. Test results consist of cyclic load-displacement curves; time-history plots of loads, displacements, accelerations, etc., during each test; analysis of peak response quantities, e.g., displacements and load-levels reached; estimated rigidities of the cladding panel-connection assembly at increasing levels of peak displacement of block cycles; as well as the relationship between drift levels and behavior of cladding panel-connection assemblies. Dynamic testing of a representative reduced scale three-dimensional model two story steel-framed building structure with and without precast concrete cladding panels, was carried out. Results provide quantitative experimental data on the earthquake resistance and stiffness of cladding connections and the overall seismic behavior of cladding connections assemblies. The test results obtained will help develop improved and more realistic analytical modeling of building structural systems interacting with heavy facades/cladding and connection systems in low/medium-rise buildings during earthquakes."

"The general objective of this research program is to document and evaluate applicable current provisions of the *Uniform Building Code* and other regulatory standards, e.g., *State of California Title 21 and Title 24, ATC 3-06, SEAOC Blue Book, Tri-Services Manual, NEHRP Guidelines,* and current practices governing the design, detailing, and installation of heavy facades/-claddings and their connections in low and medium-rise buildings with different framing systems."

In his report, Rihal listed a partial summary of representative facade and cladding types that should be considered. As noted in his Table 2.2 on "classification of building facade/building systems," these include window-wall and spandrel panels for "precast concrete cladding," "glass fiber reinforced cement (GFRC) cladding," "masonry veneer facades on framed-backing," and "stone/granite/marble facades on framed-backing." The focus here is on precast concrete cladding.

Design Issues: "Development of facade/cladding systems in buildings in seismic zones required the consideration of the following design issues: (1) facade/cladding component issues, including materials, and geometry and configuration (shape, proportion, size); (2) connections - design issues, including types of connections, location of connections, connections between cladding and structural framing and/or other cladding; and (3) supporting structural system design issues, including gravity loads and loads loads.

Overview of Informational Report Chapters: Facade/cladding performance during previous earthquakes (i.e., 1964 Alaska, 1971 San Fernando, 1978 Miyagi-ken-Oki, and 1985 Mexico City earthquakes) is presented in Chapter 5. Seismic design codes and regulations are reviewed in Chapter 6. Current design and construction practices are reviewed in Chapter 7.

Experimental Program: "In light of a general lack of test data on claddings and connections, a testing program was developed and carried out to investigate the behavior of precast concrete cladding panels with threaded-rod flexible lateral connections at top and rigid bearing connections at bottom, representative of design practices on the west coast of the U.S.A."

Objectives: "The objective of these tests was to study the static load-deflection behavior of 5/8 inch diameter threaded rods of different lengths and support conditions representative of those used in precast concrete cladding panels.

Description of Test Specimen and Test Set-Up: The test program consisted of the following: (1) Test I: testing lateral (threaded-rod) connections; (2) Test II: cyclic tests of precast concrete cladding panels and connection assembly; and (3) Test III: dynamic testing of precast concrete facade/cladding and connections in a model two-story steel moment-resisting frame structure.

For Test I, figure 1.51 (taken from fig. 1, Appendix A, Rihal [1988a]) shows the ductile connection test set-up. For Test II, figure 1.52 (taken from fig. 30, Rihal [1988a]) gives the schematic overview of the precast cladding test specimen and connections. Also for Test II, the drawings of the test set-up and specimens are found in Appendix B of Rihal [1988a]. For Test III, figure 1.53 (taken from sheet 2 of 6, Appendix C, Rihal [1988a]) if one of the drawings of the dynamic testing of precast/cladding panel and connections on a mødel two-story steel moment-resisting frame structure. The interested reader is referred to the report for the other drawings, and for the photographs, etc., that did reproduce well enough to be included here.

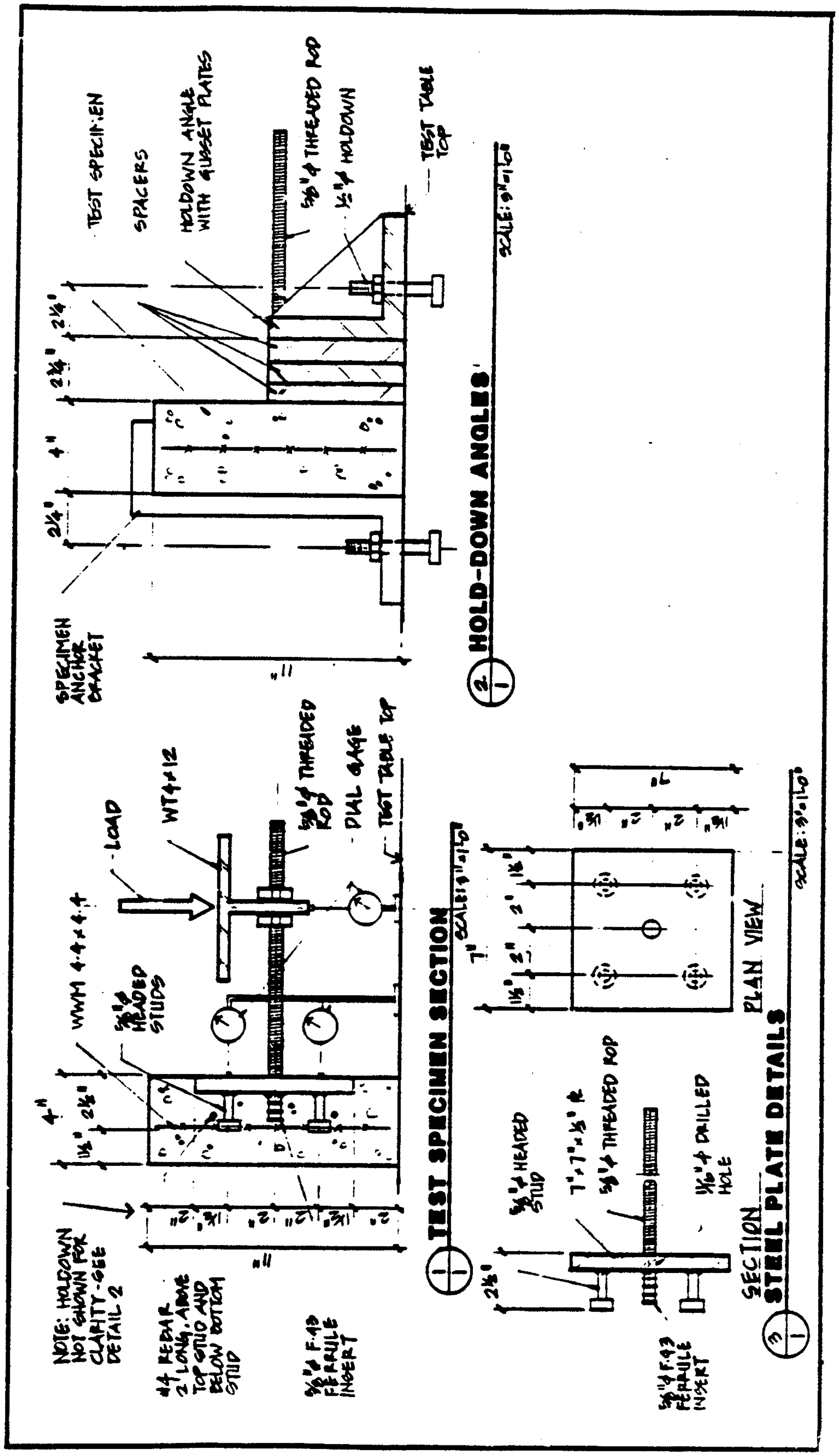


Figure 1.51. Test I: ductile connection test set-up (from Rihal [1988a]).

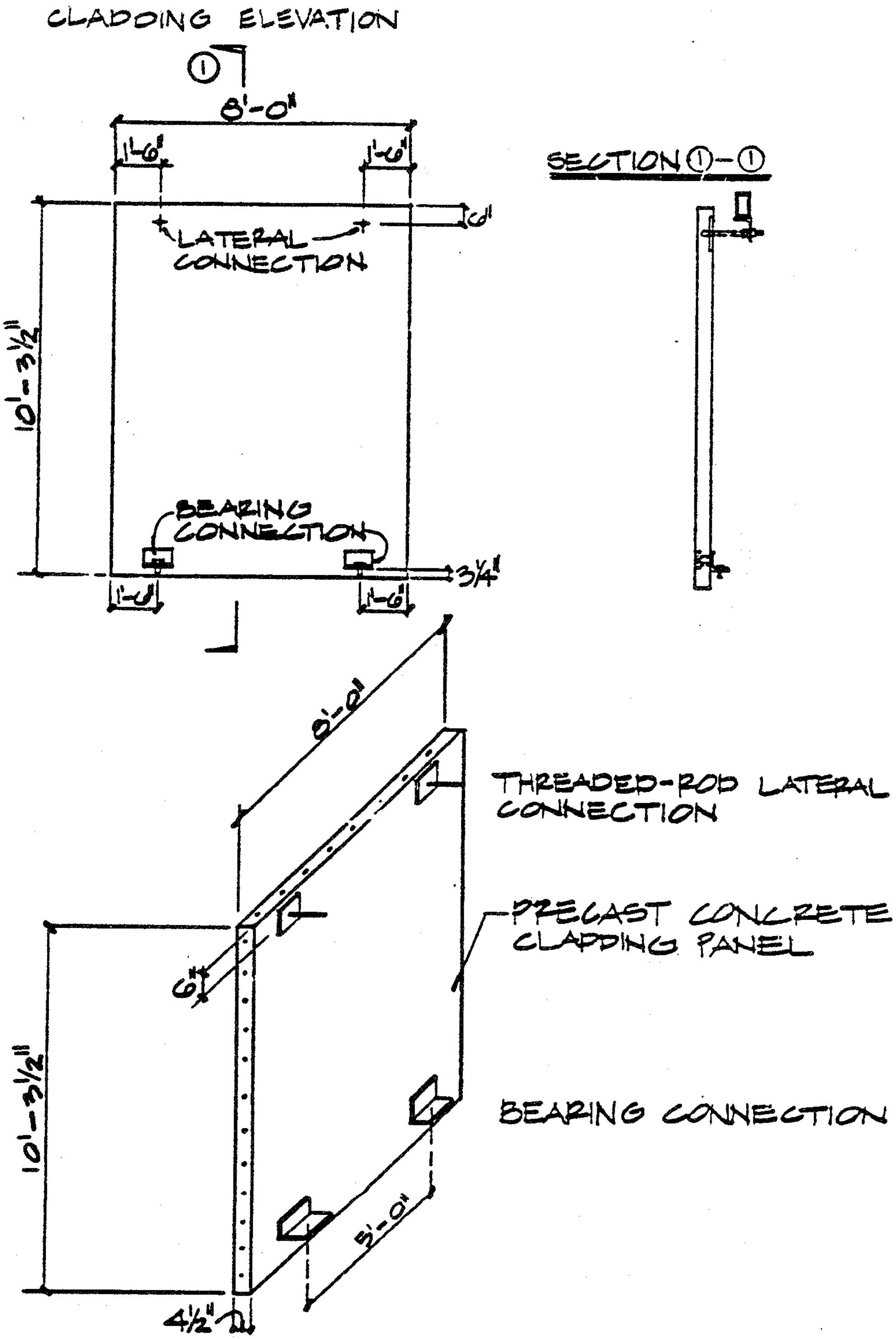


Figure 1.52. Test II: Schematic overview of precast cladding test specimen and connections (from Rihal [1988a]).

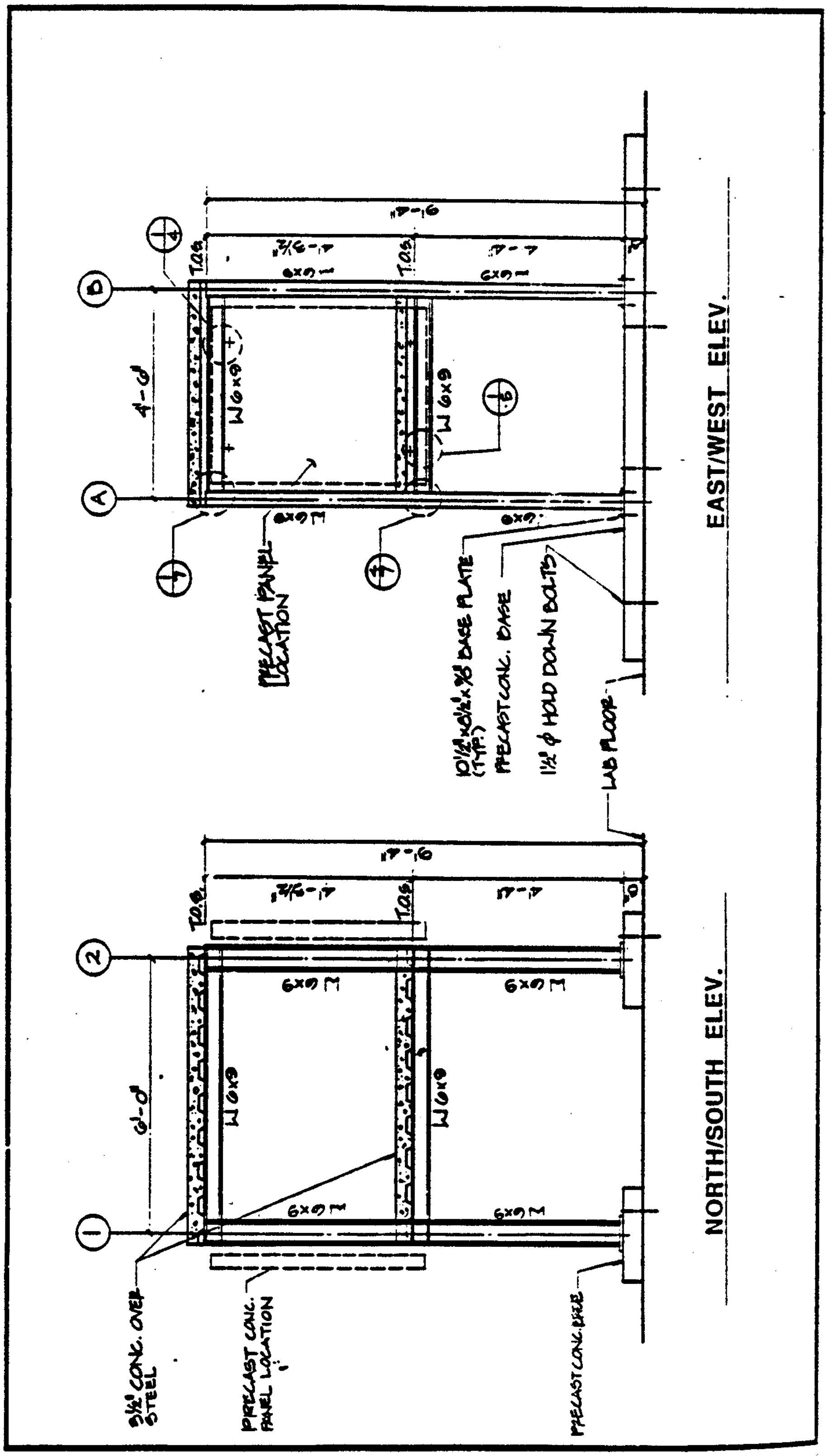


Figure 1.53. Test III: One drawing of the dynamic test set-up for precast facade/cladding panels and connections on a model two-story steel moment-resisting frame structure (from Rihal [1988a]).

Main Findings: For Test I, "Based on an experimentally obtained uniaxial tensile stress-strain curve for a 5/8 inch diameter threaded-rod, an analytical model for prediction of the load-deflection relationship for the threaded-rods tested, was developed."

For Test II, "The observed behavior and fracturing of threaded-rod lateral connection under cyclic displacements just prior to failure is shown" in the report. "Graphs of peak lateral-force resistance of threaded-rod lateral connections versus horizontal displacement (drift)" are shown in the report. "A summary of cyclic test results for the precast cladding specimens with 6-inch and 8-inch long threaded-rod lateral connections is presented in" the report. "These tables document not only the peak load and horizontal displacement (drift) levels reached, but also present estimates of service load-surcharge to the bearing angle for each of the test runs up to failure. The service-load surcharge is expressed as a percentage of the standard design load both for the bearing connection angles and the headed-studs in the bearing connections. Details of the computation of the service-load surcharge of the bearing connection due to the resistance of the threaded-rod connections are given in" the report. Figure 1.54 (taken from Fig. 23, Appendix B, Rihal [1988a]) is a graph of "peak lateral force resistance of threaded-rod connections/panel weight" versus "drift/story height."

For Test III, figure 1.55 is a sample of "typical printout of [the] spectrum analyzer display," from test run III-C in the short duration subjected to random excitation. In addition, the test results are presented in tabular form and can be found in the report.

Analytical Studies: There were three components to the analytical modeling of the behavior of cladding and connections corresponding to Tests I, II, and III, as follows: (1) behavior of threaded-rod flexible connections, (2) in-plane behavior of precast facades/claddings and connection assemblies, and (3) modal response of the two-story steel moment-resisting frame structure with and without precast concrete cladding panels. The interested reader is referred to the report.

Discussion of Results and Conclusions: For Test I, "A study of the results of Test I shows that load-capacity of threaded-rod cladding connections decreased with increasing length. Behavior of threaded-rod specimens in uniaxial tension shows evidence of strain-hardening that must be considered in design and analysis. Load-deflection behavior of cantilever threaded-rod specimens can be predicted using experimentally obtained stress-strain data with reasonably good correlation between experimental and analytical results. Simple elastic beam theory does not appear to be adequate to explain the load-deflection behavior obtained in these static tests."

For Test II, "In-plane resistance of precast concrete cladding panels is controlled by the resistance provided by the threaded-rod lateral connections at top of panels. In all cyclic test runs, failure occurred in the threaded-rods at the loading-end of top lateral connections. The levels of interstory drift that can be accommodated by the threaded-rod lateral connections can be established from the drifts at failure which varied from 0.0068H at 0.1 Hz (6-inch threaded-rod length) to 0.0117H at 0.5 Hz (8-inch threaded-rod length). Behavior of threaded-rod connections under cyclic displacements shows that further studies are needed to explain the fracturing mechanism of failure observed possibly caused by low-cycle fatigue. The lateral-force resistance offered by the threaded-rod lateral connections at the top of panels results in a service-load surcharge on the bearing connections at the bottom of the panels, which should be taken into account in the seismic design of precast concrete cladding and connections assemblies."

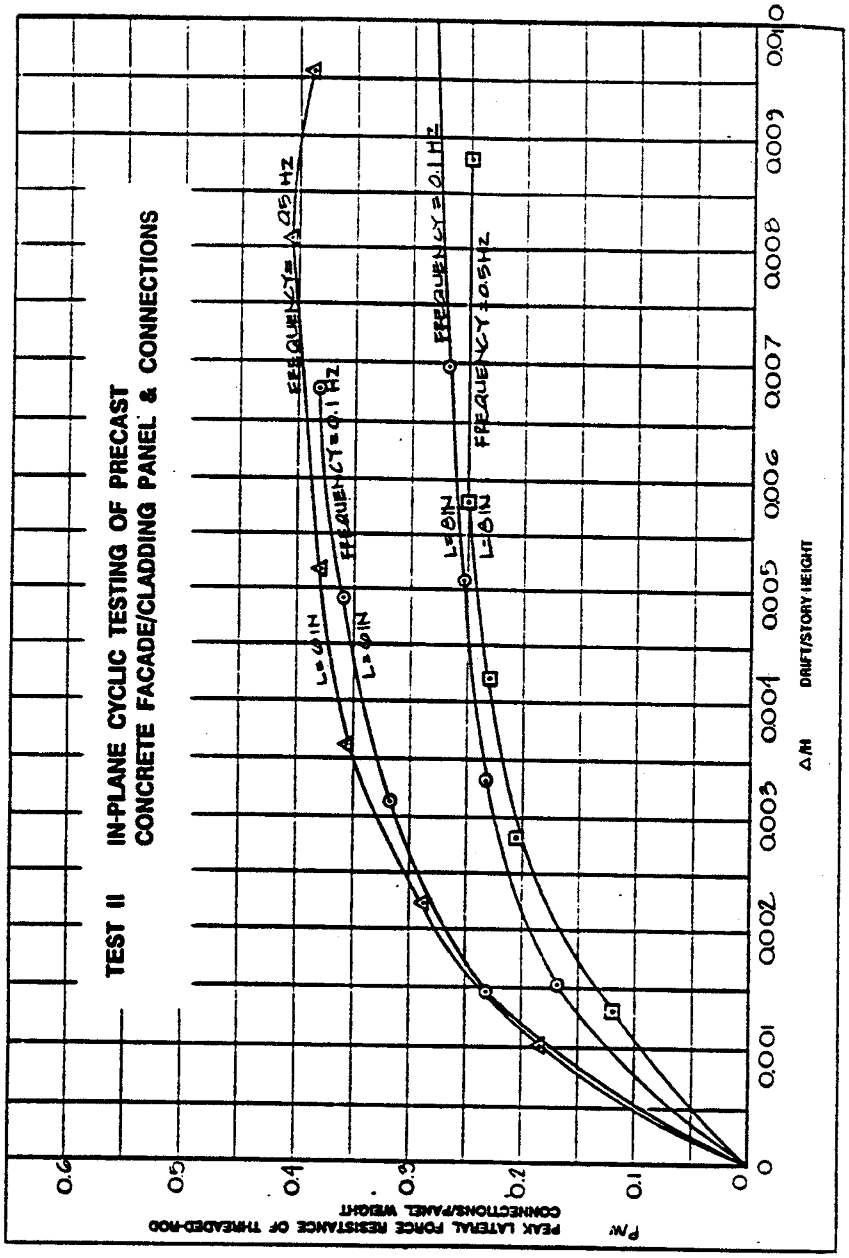


Figure 1.54. Test II: peak lateral force resistance of threaded-rod lateral connections vs. drift (from Rihal [1988a]).

For Test III, "A preliminary study of the results of shaking test carried out in Test III shows that the addition of precast concrete cladding panels to the test structure reduced the first translational mode frequency from 7 Hz to 5.9 Hz (approx. 15.71%) and the second translational mode frequency from 19.75 Hz to 17 Hz (approx. 13.92%) in the transverse direction, i.e., parallel to the plane of the cladding panels. These preliminary results show that the stiffening effects of the precast concrete cladding are significant and must be considered in the seismic design and analysis of buildings."

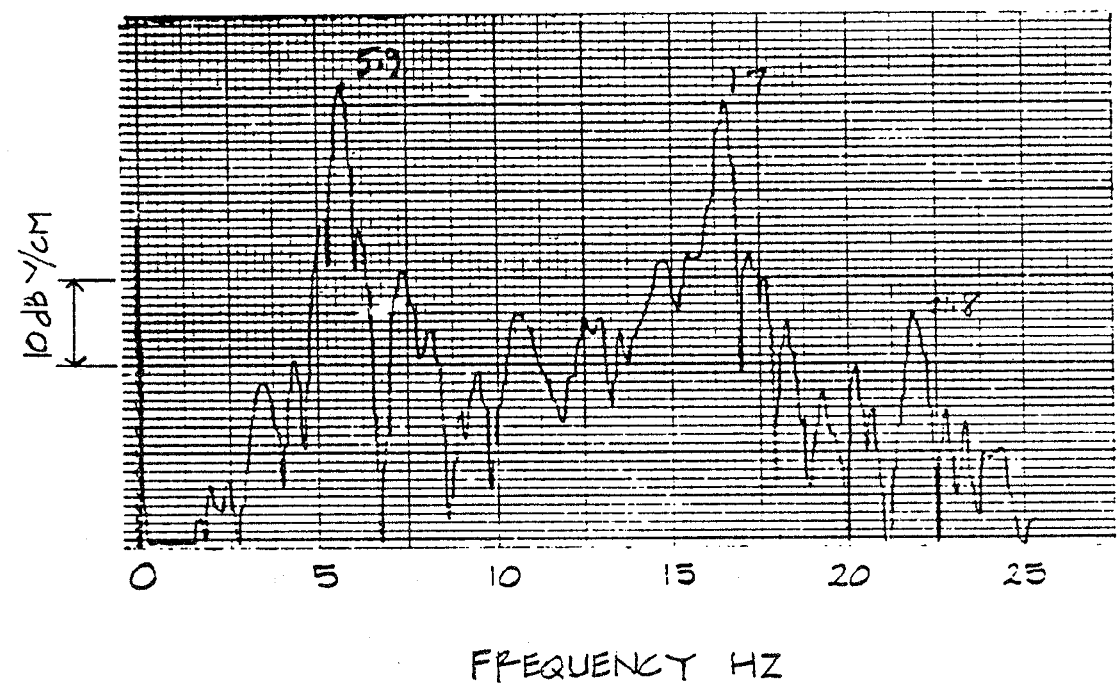


Figure 1.55. Test III: typical printout of the spectrum analyzer display from test run III-C in the short direction subjected to random excitation (from Rihal [1988a]).

CHAPTER 2

CURRENT PRACTICE FOR PRECAST CONCRETE CLADDING PANELS AND CONNECTIONS: SEISMIC ISOLATION

2.1 U.S. Codes and Interpretation

McCann [1991] summarized requirements in the 1991 *UBC* and 1990 SEAOC "Blue Book" and presents an overview as follows:

"1.3.1. Movement (drift): The code requires that both panel joints and connections allow for relative movement, or drift, between stories. If either of these [is] overlooked, the results could be catastrophic. The required allowance is the greatest of 1/2 inch, twice the wind drift, or $(3/8)R_w$, (where R_w is a numerical coefficient representing basic structural systems). In California, the [last one] usually controls, and can be up in the 3 inch range depending on how [flexible] the frame is. This movement allowance is a greater challenge, to both the architect and the connection designer, than the force. As frame analyses and designs are refined and higher strength materials are used, calculated drift often increase. This incidentally, makes it more worthwhile to utilize the panel to reduce deformations. Skill is required to get the strength and still allow the movement to isolate the panel from the building while holding it in place. The code suggests the movement be accommodated by sliding or bending of steel and that is usually what is done. Adjacent materials, such as windows, must have compatible details, which is usually left to the architect.

"1.3.2 Ductility: The code requires 'sufficient ductility' in the connection to preclude fracture of anchors or brittle failure at welds. This is largely a qualitative evaluation and a matter of judgment. The philosophy being that there should be sufficient distortion in moderate events to be a warning, and that even in large earthquakes, panels should not become detached.

"1.1.3 Strength: There are specific numerical requirements for strength. The requirements vary with the seismic zone and with different parts of the panel connection system. In Zone 4, the panel itself must resist a horizontal force of 30% of its weight; the connector body, 40%; and the fasteners, 120%. These number are not based on research, but experience has not shown them to be unsafe. It is time that they should be reviewed."

In the 1994 *UBC* (and the 1990 SEAOC "Blue Book"), the requirements (and recommendations) are as follows:

"Section 1630 - Lateral Force on Elements of Structures, Nonstructural Components and Equipment Supported by Structures.

"1630.2. Design for Total Lateral Force: The total design lateral seismic force, F_p, shall be determined from: $F_p = ZI_pC_pW_p$," where Z = seismic zone factor given in Table 16-I, I_p = importance factor given in Table 16-K, and C_p = numerical coefficient specified in Section 1630 and given in Table 16-O.

"The coefficient C_p is for elements and components and for rigid and rigidly supported equipment. Rigid or rigidly supported equipment is defined as having a fundamental period less than or equal to 0.06 second. Nonrigid or flexibly supported equipment is defined as a system having a fundamental period, including the equipment, greater than 0.06 second...

"The design lateral forces, F_p, shall be distributed in proportion to the mass distribution of the element or component. Forces, F_p, shall be used to design members and connections which transfer these forces to the seismic-resisting systems. For applicable forces in connectors for exterior panels and diaphragms, refer to Sections 1631.2.4 (on deformation compatibility) and 1631.2.9 (on diaphragms). Forces shall be applied in the horizontal directions, which result in the most critical loadings for design."

"1631.2.4. Deformation compatibility: All framing elements not required by design to be part of the lateral-force-resisting system shall be investigated and shown to be adequate for vertical load-carrying capacity when displaced $(^3/8)R_w$ times the displacement resulting from the required lateral forces. PΔ effects on such elements shall be accounted for. For designs using working stress methods, this capacity may be determined using an allowable stress increase of 1.7. The rigidity of adjoining rigid and exterior elements shall be considered as follows:

"1631.2.4.1. Adjoining rigid elements: Moment-resistant frames may be enclosed by or adjoined by more rigid elements which would tend to prevent the frame from resisting lateral forces where it can be shown that the action or failure of the more rigid elements will not impair the vertical and lateral load-resisting ability of the frame.

"1631.2.4.2 Exterior elements: Exterior nonbearing, nonshear wall panels or elements which are attached to or enclose the exterior shall be designed to resist forces, F_p, and shall accommodate movements of the structure resulting from lateral forces or temperature changes. Such elements shall be supported by means of cast-in-place concrete or by mechanical connections and fasteners in accordance with the following provisions:

1. "Connections and panel joints shall allow for a relative movement between stories of not less than two times story drift caused by wind, $(^3/8)R_w$ times the calculated elastic story drift caused by design seismic forces, of $^1/2$ inch (13 mm), whichever is greater.
2. "Connections to permit movement in the plane of the panel for story drift shall be sliding connections using slotted or oversize holes, connections which permit movement by bending of steel, or other connections providing equivalent sliding and ductility capacity.
3. "Bodies of connections shall have sufficient ductility and rotation capacity so as to preclude fracture of the concrete or brittle fracture at or near welds.
4. "The body of the connection shall be designed for $1^1/3$ times the force determined for F_p.
5. "All fasteners in the connecting system such as bolts, inserts, welds and dowels shall be designed for 4 times the forces determined for F_p.
6. "Fasteners embedded in concrete shall be attached to, or hooked around, reinforcing steel or otherwise terminated so as to effectively transfer forces to the reinforcing steel.

As noted previously, McCann [1991] stated that "in Zone 4, the panel itself must resist a horizontal force of 30% of its weight; the connector body, 40%; and the fasteners, 120%." These values are derived with $C_p = 0.75$ from Table 16-O as follows:

1. $F_p = ZI_pC_pW_p = (0.4)(1.0)C_pW_p$ for Zone 4 buildings of standard occupancy.
2. For panels: $F_p = (0.4)(1.0)C_pW_p = (0.4)(1.0)(0.75)W_p = 0.30W_p$
3. For connector bodies: $F_p = (1^1/3)[(0.4)(1.0)C_pW_p] = (1^1/3)[(0.4)(1.0)(0.75)W_p] = 0.40W_p$
4. For fasteners: $F_p = (4)[(0.4)(1.0)C_pW_p] = (4)[(0.4)(1.0)(0.75)W_p] = 1.20W_p$

Tables 16-I, 16-K, and 16-O and Section 1631.2.9 are not included here, but can be found in the 1994 *UBC*. At the time of compiling this literature survey, Table 16-O was under revision by the SEAONC Seismology Committee, but it is believed that the C_p values for cladding panels and connections will remain unchanged.

(Freeman [1989] offered an historical perspective: "It should be noted that the above forces are substantially less than the two times the weight of the precast panels criteria used in pre-1976 *Uniform Building Codes;* however, the earlier codes did not have the criteria for developing ductile type connections. The intent of the present provisions is to have the weak link in the architectural precast concrete cladding system at the ductile steel connector which can bend beyond the elastic limits, will accommodate excessive interstory movements, and will not fail in a brittle manner...")

Neither the 1994 *UBC,* the 1990 SEAOC "Blue Book," nor the 1991 *NEHRP,* require that ductility be quantified. To do so, the quotient of the maximum deformation divided by the yield deformation is needed. The yield deformation is neither specified nor not related to the story drift limit defined for serviceability in Section 1628.8. In addition, there are no requirements for the number of cycles, amplitudes of deformation, frequencies, and duration under which the connections exhibit ductile behavior.

One year after the 17 January 1994 Northridge, California, earthquake (the compilation date of this literature survey), the SEAOC Seismology Committee is re-considering the definition and use of R_w, the overstrength and ductility factor. This will affect the calculation of the inelastic interstory drift based on the elastic interstory drift for the relative movement between stories for connections and panel joints.

With respect to interstory drift, Mayes [1993] outlined design and damage control issues.

In his introduction, Mayes stated, "Traditionally, building codes have emphasized life safety as their primary goal. Because of the potential social and economic impacts of loss of function of buildings, there is a new focus on the control of damage in seismic design. Although there are multiple components in the control of damage, this paper focuses on one aspect known to be as significant factor - and that is *interstory drift.* Where post-earthquake functionality is part of the owner's performance criteria, it is necessary to consider the structure's interstory drift in determining the level and type of damage which can be expected," as well as the deformations that must be accommodated by the cladding connections.

Mayes continued, "...A key issue in limiting drift-related damage is to obtain the best estimate of the displacements and interstory drifts associated with the nonlinear force-deflection characteristics of the structure. The (1991) *UBC* attempts to address this by multiplying the displacements and drifts associated with the design base shear, V_b, by the arbitrary factor, $^3/_8 R_w$. The 1990 SEAOC 'Blue Book' Commentary acknowledges that this may underestimate the realistic drift and a more appropriate factor may be as high as R_w. More accurate solutions for the drift calculation can be obtained through considerations of nonlinear time-history analysis procedures, although this is seldom done."

Porush [1992] presented an overview of the current building code seismic requirements for nonstructural elements. His paper included the requirements of the 1988 *UBC*, 1990 SEAOC "Blue Book," and the 1988 *NEHRP* Recommended Provisions for the Development of Seismic Regulations for Buildings. The 1988 *UBC* may be too old for consideration, here, but his paper provides some historical information that forms the basis of more current codes, such as the 1994 *UBC* and the 1991 *NEHRP* Provisions.

The 1991 *NEHRP* contains recommended provisions for architectural component design in Section 8.2 as follows:

"8.2.1. General: Systems or components listed in Table 8.2.2 and their attachments shall be designed and detailed in accordance with the requirements of this chapter. The design criteria for systems or components shall be included as part of the design documents." In Table 8.2.2, the architectural component seismic coefficient, C_c, and the performance criteria factor, P, are given. For exterior nonbearing walls, $C_c = 0.9$, and $P = 1.5$ for seismic hazard exposure groups I, II, and III.

"8.2.2. Forces: Architectural components and their means of attachment shall be designed for seismic forces (F_p) determined in accordance with the following equation: $F_p = A_v C_c P W_c$, where

F_p = the seismic force applied to a component of a building or equipment at its center of gravity,

A_v = the seismic coefficient representing effective peak velocity-related acceleration from Sec. 1.4.1, and

W_c = the weight of the architectural component.

"The force (F_p) shall be applied independently vertically, longitudinally, and laterally in combination with the static load of the element.

"Exceptions: When positive and negative wind loads exceed F_p for nonbearing exterior walls, these loads shall govern the design...

"8.2.3. Exterior Wall Panel Connections: The connections of exterior wall panels to the building seismic resisting system shall be design for the design story drift as determined in Sec. 4.6.1. (on story drift determination) or in accordance with Sec. 5.6 (on modal forces, deflections, and drifts) or 5.8 (on design values for the modal base shear).

"8.2.4. Architectural Component Deformation: Architectural components shall be designed for design story drift of the structural resisting system as determined in accordance with Sec. 4.6.1 or Sec. 5.8. Architectural components shall be designed for vertical deflection due to joint rotation of cantilever structural members."

"4.6.1. Story Drift Determination: The design story drift (Δ) shall be computed as the difference of the deflections at the top and bottom of the story under consideration. The deflections of Level x at the center of the mass (δ_x) shall be determined in accordance with: $\delta_x = C_d\delta_{xe}$, where C_d = the deflection amplification factor in Table 3.3, and δ_{xe} = the deflections determined by an elastic analysis."

"The elastic analysis of the seismic force resisting system shall be made using the prescribed seismic design forces of Sec. 3.4.2.

"For determining compliance with the story drift limitation of Sec. 3.7, the deflections of level x at the center of mass (δ_x) shall be calculated as required in this section. For purposes of this drift analysis only, it is permissible to use the computed fundamental period (T) of the building without the upper bound limitation specified in Sec. 4.2.2 when determining drift level seismic design forces.

"Where applicable, the design story drift (Δ) shall be increased by the incremental factor relating to the P-delta effects as determined in Sec. 4.6.2."

For tables and sections cited but not included herein, the interested reader is referred to the 1991 *NEHRP*. The commentary of 1991 *NEHRP* contains useful information and design guidance for all sections cited above.

Bachman and Drake [1994] presented information on the 1994 *NEHRP* provisions for architectural, mechanical and electrical components. Their abstract is as follows: "The force equations presently used for the seismic design of nonstructural components vary from code to code. A comparison of the force equations used in the 1991 *UBC* and the 1991 *NEHRP* Provisions indicates that the *NEHRP* values are larger in the high seismic zones and the *UBC* values are larger in the low seismic zones. Neither code considers soil effects or seismic relative displacements for nonstructural components. This paper discusses newly developed force and displacement equations proposed for incorporation into the 1994 *NEHRP* Provisions and compares them with current code provisions."

The revision objectives stated by Bachman and Drake are noted as follows: "Seismic force equations for nonstructural components have been proposed for incorporation into the 1994 *NEHRP* Provisions that consider the following: (1) Component weight and mass distribution, including dynamic properties. Both the 1991 *UBC* and the 1991 *NEHRP* Provisions consider this; (2) Location of structure within regional seismic zone. Both the 1991 *UBC* and the 1991 *NEHRP* Provisions consider this; (3) Seismic response of the primary supporting structure to earthquake input motions, including site effects. Neither the 1991 *UBC* and the 1991 *NEHRP* Provisions consider this; (4) Location of component within structure. The 1991 *UBC* considers this to a limited extent; (5) The safety hazard which would result should the component separate from structure. Both the 1991 *UBC* and the 1991 *NEHRP* Provisions indirectly consider this; (6) Importance of component function to operation of facility. Both the 1991 *UBC* and the 1991 *NEHRP* Provisions consider this, but neither assure function; (7) Component anchorage ductility and energy absorption capability. Neither the 1991 *UBC* and the 1991 *NEHRP* Provisions consider this. Additional desirable attributes of the proposed force equations include: (1) Input accelerations for components which rationally reflect actual structural accelerations at the component attachment point(s); (2) Input accelerations for components which are consistent with the input design ground motion at grade level; and (3) Input acceleration for grade-level components should match grade level design ground motion accelerations used in the design of the structure."

Bachman and Drake continued by presenting the force equations. "The following seismic force equations are proposed for nonstructural components. To meet the need for a simple, easy to use force equation, a default equation for the seismic design force (F_p) is provided first,

$F_p = 4.0A_gI_pW_p$ (1994 *NEHRP* equation 3-1).

Or alternatively, the following more complex equations may be used. These equations will generally yield smaller design values for F_p than equation 3-1. The value of F_p obtained from equation 3-2 need not exceed the value obtained from equation 3-1."

$F_p = a_pA_pI_pW_p / R_p$ (1994 *NEHRP* equation 3-2).

$A_p = A_g + (A_r - Ag)(x/h)$ (1994 *NEHRP* equation 3-3).

$A_r = 2.0A_s \leq 4.0A_g$ (1994 *NEHRP* equation 3-4).

$F_p(\text{minimum}) = 0.5A_gI_pW_p$ (1994 *NEHRP* equation 3-5).

The terms used in the 1994 *NEHRP* equations 3-1 to 3-5 are defined as follows:

A_a = Effective peak site acceleration (take from Design Value contour maps provided in the *NEHRP* Provisions).

A_g = Component acceleration at base of structure (ground), equals effective peak acceleration including site effects, $A_g = F_sA_s$.

A_p = Component acceleration at point of attachment to the structure.

A_r = Component acceleration at structure root (or highest) level of primary lateral force resisting system.

A_s = Structural response acceleration, $A_s = F_vA_v / T^{2/3} \leq 2.5A_g$.

A_v = Velocity-related site acceleration (taken from Design Value contour maps provided in the *NEHRP* provisions).

a_p = Component amplification factor, varies from 1.00 to 2.50 (values are tabulated for each component)

F_a = Site coefficient at 0.3 second period (function of A_s).

F_p = Seismic design force applied to a component at its center of gravity.

F_v = Site coefficient at 1.0 second period (function of A_v).

h = Roof elevation of structure.

I_p = Component importance factor, varies from 1.00 to 1.50 (values are tabulated for each component)

R_p = Component response modification factor, varies from 1.5 to 6.0 (values are tabulated for each component).

T = Effective fundamental period of the structure.

W_p = Component maximum operating weight.

x = Elevation in structure of component anchorage.

"The seismic design force (F_p) is to be applied independently vertically, longitudinally, and laterally in combination with the static loads on the component. The seismic design force (F_p) is dependent upon the weight of the component, the component amplification factor, the component acceleration at the point of attachment to the structure, the component importance factor, and the component response modification factor. A lower limit for F_p is defined in equation 3-5 to assure a minimum seismic design force. To meet the need for a simpler formulation, a conservative maximum value for F_p is defined as equation 3-1. This maximum value can always be used if convenient to the designer, or if the component anchorage must be designed before information on the structure is available."

The interested reader is referred to the Bachman and Drake paper for further information on the component amplification factor, component acceleration factor, component importance factor, component response modification factor. In addition there is a section on seismic relative displacements, that includes information on revision objectives and relative displacement equations that are "based on either the building structure analysis or the building drift limitations," as well as a final section on summary and conclusions.

2.2 Foreign Codes

The IAEE [1992] has compiled earthquake regulations for thirty-seven countries. "The IAEE Central Office has made every effort to obtain English translations of current codes and regulations. Some have been translated by the Central Office from the official languages of the countries into English, which is the official language of the IAEE. From the 1988 edition, we collected the brief description of each design code, including the allowable strength of materials, and put it at the beginning of each section. Unfortunately, the uniformity in language and appearance has not been fully achieved in the present edition."

The thirty-seven countries included in the IAEE [1992] are as follows: Algeria, Argentina, Australia, Austria, Bulgaria, Canada, Chile, (People's Republic of) China, Colombia, Costa Rica, Cuba, Egypt, El Salvador, Ethiopia, France, (formerly Republic of) Germany, Greece, India, Indonesia, Iran, Israel, Italy, Japan, Mexico, New Zealand, Nicaragua, Peru, Philippines, Portugal, Romania, Spain, Switzerland, Turkey, (former) Union of Soviet Socialist Republics, United States of America, Venezuela, and (former) Yugoslavia.

This literature survey includes information from post-1970 codes on the strength and deformation criteria used to design and detail heavy cladding panels and their panel-to-frame connections in countries with seismic zones similar to Zones 4 and 3 as defined by the *Uniform Building Code* [1994]. The interested reader is referred to IAEE [1992] for information on the seismic-resistant design of the structural framing system, and on the codes (noted below) as not being included herein.

The 1961 Austrian code and the 1964 Cuban code are too old to be included.

The following codes are presented in languages other than English and are not included: the 1988 Algerian code (in French), the 1987 El Savadoran code (in Spanish), the 1986 Italian code (in Italian), the 1983 Nacaraguan code (in Spanish), the 1992 Spanish code (in Spanish), and (undated) Soviet Union code (in Russian).

The following codes as included in the IAEE [1992] do not specifically mention cladding panels or connections but may include qualitative descriptions of design intentions: 1987 Bulgarian code, 1972 Chilean code, 1989 People's Republic of China code, 1986 Costa Rican code, 1990 French code, 1981 German code, 1984 Greek code, the (undated) Peruvian code (with an equation non-structural elements and anchorages, but no quantification of terms), 1983 Portuguese code, 1981 Romanian code, 1989 Swiss code, 1975 Turkish code, and (undated) Venezuelan code.

The latest U.S. codes are summarized in the previous section.

The Argentine code, Regulations for Antiseismic Design in Argentina, is published by the Center for Study on Structural Norms for Concrete. Cladding is not specifically mentioned. Under "Parts of the Construction," it is noted that "each element or part of the construction should be joined directly or indirectly to the main structure in order to transmit the seismic forces." Equations are given "to check the stability and anchorage of the elements or parts of the construction located at the level i," but cladding is not explicitly listed. Exterior and interior walls, partitions and fence walls with height of more than 2m are listed, but these are not cladding panels.

The 1979 Australian code, Australian Standard 2121 for the Design of Earthquake-Resistant Buildings, is published by the Standards Association of Australia. In Section 7, minimum earthquake forces for parts of buildings are given as the horizontal force factor C_p for parts of buildings. In any horizontal direction, the value of $C_p = 2.00$ for connections for exterior panels or for elements complying with Clause 8.3. Clause 8.3 is for exterior elements and includes, "Non-structural elements which are attached to or enclose the exterior of a building shall be capable of accommodating movements of the structure resulting from the horizontal earthquake forces as follows: (a) All connections and panel joints shall allow for a relative movement between stories equal to (3.0/K) times the story drift calculated from the horizontal forces prescribed by this standard, or 6 mm, whichever is greater; (b) Connections shall have sufficient ductility and rotation capacity to preclude brittle failure at or near welds or fracture of the concrete. Inserts in concrete shall be attached to or hooked around reinforcing steel, or otherwise terminated so as to transfer forces effectively to the reinforcing steel; and (c) Connections to permit movements in the plane of the panel shall include properly designed sliding connections using slotted or oversize holes, or connections which permit movement be bending of steel, or other suitable connections which have been proved to be adequate. The minimum permissible value of the horizontal force factor K is given for specific structural systems.

The 1990 Canadian code, the National Building Code of Canada, is published by the Associate Committee on the National Building Code of the National Research Council of Canada. "Parts of buildings and their anchorage shall be designed for a lateral force, V_p, equal to vS_pW_p, distributed according to the distribution of mass of the element under consideration, where v (zonal velocity ratio = the specified zonal horizontal ground velocity expressed as a ratio to 1 m/s) is determined in Subsection 2.2.1 (not given in IAEE [1992]), except when Z_v (velocity-related seismic zone) equals zero and Z_a (acceleration-related seismic zone) is greater than zero, v shall be taken as 0.05." The values of S_p are 1.5 normal to flat surface of all exterior and interior walls, and 15 in any direction for connections/ attachments.

Bruneau and Cohen [1994] reviewed the National Building Code of Canada (NBCC) earthquake-resistant design requirements of cladding connectors. In the abstract, the authors wrote, "As a consequence of the recent increases in the severity of seismic design force requirements in Canada, practicing engineers who design cladding connectors should be concerned with their seismic resistance. The current design requirements of the 1990 edition of the NBCC for nonstructural components call for unduly high prescribed design forces for the cladding connectors without providing justification, commentary, or substantiation for this constraint, nor guidance on how this is to be achieved. This paper offers some rationalizations of the current design approach, recommends possible abatements of the requirements in special cases, and points toward future directions and alternate philosophy for the design of cladding connectors. In particular, the following are recommended: (i) the scope of Part 4 of the NBCC should be modified to specifically indicate that cladding connectors are to be designed by a professional engineer, (ii) the latest cladding-connector seismic-resistant design philosophy of the Structural Engineers Association of California should be incorporated into the NBCC; (iii) a distinction should be made between out-of-plane and in-plane cladding-connector seismic-resistant design requirements; (iv) a commentary should be written on cladding-related seismic-resistant design issues to clearly state current philosophy, uncertainties, and limits of knowledge be included in the building code, and (v) standardized seismic-resistant cladding connectors (should) be developed with capacities to meet prescribed levels of ductile behavior and interstory drifts and widely distributed to the profession."

The 1981 Colombian code, Standard AIS 100-81 Seismic Requirements for Buildings, is published by the Asociacion Colombiana de Ingenieria Sismica. Chapter 6 is on "Requirements for Non-Structural Elements." "Non-structural elements shall be designed to resist seismic forces determined in accordance with the following formula: $F_p = A_v C_c W_c$, where F_p = the seismic force applied to the element at its center of gravity; C_c = the seismic coefficient for the component (such as C_c = 0.9 for exterior non-bearing walls, C_c = 3.0 for wall attachments); and W_c = the seismic coefficient representing the effective peak velocity-related acceleration" [given elsewhere]. "Attachment of exterior wall panels to the building seismic resisting system shall have sufficient ductility and provide rotational capacity needed to accommodate the design story drift" [determined elsewhere, but 'shall not exceed 1.5 percent of the story height for any story of the building']. "Provisions shall be made in the non-structural element for the design story drift as determined" [elsewhere]. "Non-structural walls shall be anchored to the roof and all floors which provide lateral support to the wall. The anchorage shall provide a direction between the walls and the roof or floor construction. The connection shall be capable of resisting a lateral seismic force F_p, induced by the wall, but not less than a force 1500 A_v (kg) per linear meter of wall. Walls shall be designed to resist bending between anchors where the anchor spacing exceeds 1.20 meters."

The 1988 Egyptian code, Regulations for Earthquake-Resistant Design of Buildings in Egypt, is published by the Egyptian Society for Earthquake Engineering. In the section on "Parts or Portions of Buildings," "Any part or portion of a building shall be designed for a seismic force F_p applied at its center of mass in each direction under consideration as given by: $F_p = C_pW_p$, where C_p is the seismic design coefficient for a part or portion of a building, and W_p is the weight of a part or portion of a building. The seismic design coefficient for a part or portion of a building C_p is determined from: $C_p = C_sP_pR_p$, where C_s is the seismic design coefficient determined [elsewhere], P_p is the position for a part or portion of a building, and R_p is the risk factor for a part or portion of a building. The position factor P_p reflects the amplification of the ground motion by the structure supporting the particular component and is determined from: $P_p = 1.0 + h_p/H$, where h_p is the height at which the part of portion of a building is located, and H is the total height of the building. The risk factor R_p for a part or portion of a building, an allowance for its performance during and immediately following an earthquake," is given in a table within the code. Under "deformation due to earthquake loads," "the inter-story deflection shall not exceed 0.005 times the story height nor shall exceed 2.0 cm."

The 1983 Ethiopian code, ESCP1: 1983, Code of Practice for Loading, is published by the Ministry of Urban Development and Housing. In the section on "lateral force on elements of structures," "Architectural systems and components and their attachments shall be designed for seismic forces determine from: $F_c = \alpha\beta_cG_c$, and distributed according to the distribution of mass of the element under consideration, where F_c is the seismic force applied to a component of a building or equipment at its center of gravity; α is the design bedrock acceleration (given by an equation); β_c is the seismic coefficient for components of architectural system (2 for exterior and interior walls, and 20 for connections for exterior and interior walls, except those forming part of the main structural system); and G_c is the weight of a component of a building or equipment." Under "special requirements for nonstructural components," "Nonstructural components shall be designed so as not to transfer to the structural system any force unaccounted for in the design, and any interaction of rigid elements such as walls and the structural system shall be designed so that the capacity of the structural system is not impaired by the action or failure of the rigid elements."

The 1983 Indonesian code, The Indonesian Earthquake Code, is published by the Ministry of Public Works. The requirements for the separation between the structural element and the nonstructure, "precast concrete claddings and other claddings of similar mass," shall be as follows: "(a) Ratio of interstory deflection to story height not exceeding 0.0003; No requirements for separation; (b) Ratio of interstory deflection to story height exceeding 0.0003 but not greater than 0.005 or 2 cm. Elements shall be positively separated from the structure so as to allow the structure to deform four times that calculated in accordance with Clause 3.6.1 without the elements coming into contact with the structure or with adjacent elements. A minimum separation of 1 cm shall be maintained between the structure and the vertical surfaces of the element. Construction tolerances shall not reduce the required separations." Clause 3.6.1 for computed deformations due to earthquake loads states "Structural deformations shall be calculated from the loads determined [elsewhere] multiplied by the factor CI/C_d for the equivalent static load analysis, or $CI/0.9C_d$ for a dynamic analysis." The value of C is the basic seismic coefficient for the full design procedure; I is the importance factor of the building; and C_d is the basic seismic coefficient due to importance and structural type factor. In Section 4.2 on connections, it is stated that "All elements, components or equipment... shall be positively connected to the structure to resist the specified seismic loads. Friction due to gravity load shall not be used to provide the required resistance to horizontal loads. Connectors to... exterior panels including anchor bolts shall be corrosion-resisting and ductile, with adequate anchorages. In the case of precast concrete panels, anchorages shall be attached to, or hooked around, panel reinforcing." In section 4.5 on design loads, "All elements and components shall be designed for a seismic load F_p in the direction specified as given by $F_p = C_pK_pPW_p$, where W_p is the weight of the element, component or equipment, and C_p is equal to C which is a function of element period for elements supported by the structure C_p is equal to C_d for the structure." "The K_p factor reflects the distribution and amplification of the ground motion by the structure supporting the particular component. $K_p = 1.0 + h_i/H$, where h_i is the height at which the element or component is located and H is the overall height of the building. K_p shall be taken as 1.0 for structures supported directly on the ground. The K_p factor for more important elements of the structure shall be calculated form a more exact analysis." "The P factor is an allowance for both the estimated performance of the element or component and the importance of its performance during and immediately following an earthquake. If the period of an element is close to that of the structure considerable amplification may occur. If the ratio of the period of the structure to that of the elements is between 0.6 and 1.4 the value of P shall be multiplied by 2 unless a special analysis is carried out." The value of P is equal to 8 for "veneers, exterior prefabricated panels and ornamental appendages, and their connections."

The 1988 Iranian code, the Iranian Code for Seismic Resistant Design of Buildings, is published by the Building and Housing Research Center. In the "story drift" section, "Lateral displacement at each level of the building in relation to the upper or the lower level, which is calculated by taking into account the lateral forces jointly with the torsional moment, shall not exceed 0.005 of the height of the building." In the section on seismic lateral force on building components and added portions, "Building components and portions added to the building shall be designed against the lateral force which is obtained from $F_p = AB_pIW_p$, in which A and I are the values" of design base acceleration and the importance factor of the building, respectively, "which have been used for the design of the entire building, W_p is the weight of the building component or of the added part and portion under consideration." For outside and inside ornamental elements or components of the building, B_p is 2 in any direction. For connections of prefabricated structural elements, B_p is 1 in any direction.

The 1975 Israeli code with a 1990 amendment, the Characteristic Loads in Buildings: Earthquake - 197 (Israel Standard SI 413), Amendment Sheet No. 3 of September 1990 including Annex C, is published by The Standards Institution of Israel. In the section on equivalent forces acting on structural components, it is stated that "In addition to the analysis of the structure as a whole, the loads being the equivalent forces according to the preceding clauses, the safety of the structural components shall be examined as follows: (a) The different parts of the structure shall be acted upon by equivalent horizontal forces by formula: F = CW, where C is 0.20 for external and internal load-bearing and non-load-bearing walls, partitions, fences, etc., in the direction perpendicular to the plane of the structural component, C is 2.00 for the joints of external wall panels in any direction, C is 1.00 for internal and external decorative elements in any direction, and W is the vertical load on the component of the structure calculated by W = G + kQ, where G = dead load, Q = live load, and k = incidence factor for different types of structures."

The 1986 Italian code, Norme tecniche relative alle construzioni sismiche-1986, is published by the Minstero del Lavori Pubblici. The code is printed in Italian and not included here.

The Japanese code, Standard for Aseismic Civil Engineering Constructions, Earthquake-Resistant Design Method for Buildings, is included in IAEE [1992], but without a date. In Part 2 on Earthquake-Resistant Design Method for Buildings, the section on story drift states, "The drift of each story of the building caused by lateral seismic shear for moderate earthquake motions... shall not exceed 1/200 of the story height. This values can be increased to 1/120, if the nonstructural members shall have no severe damage at the increased story drift limitation." Part 2 that was included in IAEE [1992] does not contain any information on cladding panels and cladding connections.

The 1987 Mexican code, Complementary Technical Normas for Earthquake Resistant Designs, is published by the Departamento del Distrito Federal. "Partitions, curtain and exterior walls shall be considered as follows: Where walls do not contribute to resist lateral forces, they shall be attached to the structure so that the structure deformation in the plane of walls is not restricted by them. Preferably these walls shall be made of very flexible or weak materials." There is no other information on cladding panels and cladding connections. "Differences of lateral displacements in consecutive stories caused by interstory shear forces obtained with any of the seismic analysis methods... of this code, shall not exceed 0.006 times the difference of their corresponding heights. This limit will not be valid where elements unable to withstand considerable deformations, such as masonry walls, are properly separated from the main structure in such a way that they will not be damaged by the drift of a structure. In the case, the interstory drift limit will be 0.012. Computation of lateral deflections can be omitted when using the simplified seismic analysis method."

The 1992 New Zealand code, the Code of Practice for General Structural Design and Design Loadings for Buildings (New Zealand Standard NZS 4203:1992), is published by the Standards Association of New Zealand. Information pertinent to cladding panels and connections is included here. For further information on the sections cited but not included here (such as on analysis methods, etc.) and for information on framing design, the interested reader is referred to the code.

Part 4 contains the Earthquake Provisions.

Section 4.12.1 contains General requirements.

Section 4.12.1.1 on Requirements for Parts states, "All parts of structures, including permanent non-structural components and their connections, and the connections for permanent services equipment supported by structures shall be designed for the seismic forces specified herein." The value of the risk factor P.I parts, the failure of which could cause a life hazard, is equal to 1.10.

Section 4.12.1.2: "Except as otherwise determined, the seismic force on parts of structures for the serviceability or ultimate limit state as appropriate shall be determined from a design coefficient, C_{pi}, calculated in accordance with 4.12.2."

Section 4.12.1.3: "The design forces on a part of a building may be determined from an analysis of the response of the part performed in accordance with established principles of structural design. The response of the part shall be the total response at the level of the part (i.e., with the ground motion added to the structural motion relative to the ground). The analysis shall include modelling of the connections of the part to the structure and allowance for potential overstrength of the structure and of the part. If a modal response spectrum analysis is used this shall be carried out in accordance with Section 4.9. If a numerical integration time history analysis is used this shall be carried out in accordance with Section 4.10. The resulting seismic coefficient may be substituted for the specified values of C_{ph} or C_{pv}," where C_{ph} is the basic seismic coefficient for a part and C_{pv} is the basic vertical seismic coefficient for a part.

Section 4.12.1.4: "The horizontal seismic force on parts, F_{ph}, shall be determined from $F_{ph} = C_{ph}W_pR_p$, where C_{ph} shall be taken equal to C_{pi} at the level of the part from 4.12.2.3, and W_p is the weight of the part." R_p is the risk factor for the part.

Section 4.12.1.5 is on horizontally cantilevered parts.

Section 4.12.1.6: "Connections for parts shall be designed for seismic forces corresponding to a structural ductility factor for the part, μ_p, equal to 1.0, unless a capacity design is employed to demonstrate that a greater ductility factor is achievable. Where, in the event of failure of connections, there is a risk to persons, design forces on connections shall be multiplied by 1.5 or the connection shall be detailed for displacement ductility factor of not less than 2.0."

Section 4.12.1.7: "Deflections of parts under the prescribed seismic forces shall be limited so as not to impair their strength or function, or lead to damage to other building components."

Section 4.12.1.8: "Connections between the parts and the building structure shall be designed to accommodate the interstory deflections determined in accordance with 4.7.4."

Section 4.12.2 contains Basic Horizontal Coefficients for Parts.

Section 4.12.2.1 is the General section: "The basic horizontal coefficient for a part at level i, C_{pi}, shall be determined form the floor coefficient at level i, C_{fi}. C_{fi} is determined from 4.12.2.2 and C_{pi} is determined from it in accordance with 4.12.2.3."

Section 4.12.2.2 is on the Floor Coefficient: "The floor coefficients at and below the base of the structure, C_{fo}, and at the level of the uppermost principal seismic weight, C_{fn}, shall be as given (in the following three equations). The floor acceleration coefficient at levels between the vase and the level of the uppermost principal seismic weight, C_{fi}, shall be determined by either method (a) or (b) below. For levels other than at floors, linear variation of C_{fi} between adjacent floor levels may be assumed.

$C_{fo} = 0.4RZL_s$ for the serviceability limit state

$= 0.4RZL_u$ for the ultimate limit state

$C_{fn} = \{[C_b(T_1,\mu_0)] / [C_b(T_1,\mu)]\} \times \{F_n/W_n\}$ "

where $\mu_0 = \mu = 1.0$ for the serviceability limit state. For the ultimate limit state, μ_0, is the structural ductility factor that would apply to the building structure calculated with overstrength, and shall be taken as 1.0 unless capacity design is applied to the building structure to justify a larger value." R is the risk factor for a structure. Z is the zone factor. L_s and L_u are the limit state factors for the serviceability and ultimate limit states, respectively. T_1 is the fundamental translational period of vibration. $C_b(T,\mu)$ is the basic seismic acceleration coefficient which accounts for different soil conditions, structural ductility factors, μ, and fundamental translational periods of vibration, T_1. F_n is the inertia force at the height of the uppermost principal seismic weight, h_n, used for the design of the structure, either the equivalent static force or the force from the combination of modal forces, as appropriate to the method of analysis used for the building structure.

(a) "Where the equivalent static method is used for the floor coefficient shall be as given:

$C_{fi} = \{[C_b(T_1,\mu_0)] / [C_b(T_1,1)]\} \times \{C_{f0}[1\text{-}h_i/h_n] + C_{fn}[1\text{-}h_i/h_n]\}$

(b) Where the modal response spectrum method of analysis is used the floor coefficient shall be as given:

$C_{fi} = \{[C_b(T_1,\mu_0)] / [C_b(T_1,\mu)]\} \times \{F_i/W_i\}$ "

Section 4.12.2.3 is on the Basic Horizontal Coefficient. "The basic horizontal coefficient for parts at level i, C_{pi}, shall be given by:

$C_{pi} = C_b(T_{pe}, \mu_p)\ (C_{fi}/0.4)$

where $C_b(T_{pe}, \mu_p)$ is the basic seismic acceleration for intermediate soil and T_{pe} is the equivalent period of the part given by $= 0.2T_p/T_1$ but not to be taken less than 0.4s."

The Philippine code, the National Structural Code for Buildings, Chapter 2 on Lateral Forces, is published by the Republic of the Philippines. Under the section on "exterior elements," "Precast or prefabricated nonbearing nonshear wall panels or similar elements which are attached to or enclose the exterior shall be designed to resist the forces determined from," $F_p = ZIC_pW_p$ (similar to the *UBC)*, where $C_p = 0.30$ in any direction for "connections for prefabricated structural elements other than walls, with the force applied at the center of gravity of the assembly." "The force shall be resisted by positive anchorage and not by friction." The elements "shall accommodate movement of the structure resulting from lateral forces or temperature changes. The concrete panels or other similar elements shall be supported by means of cast-in-place concrete or mechanical connections and fasteners in accordance with the following provisions: Connections and panel joints shall allow for a relative movement between stories of not less than two times story drift caused by wind or (3.0/k) times the calculated elastic story displacement caused by required seismic forces, or 12 mm, whichever is greater." The value of K is "the horizontal force factor for buildings," that corresponds to the "type or arrangement of resisting elements" (as in the older versions of the *UBC)*. "Connections to permit movement in the plane of the panel for story drift shall be properly designed with sliding connections using slotted or oversized holes, or connections which permit movement by bending of steel of other connections providing equivalent sliding and ductility capacity." "Bodies of connections shall have sufficient ductility and rotation capacity so as to preclude fracture of the concrete or brittle failures at or near welds." "The body of the connector shall be designed for $1^{1}/3$ times the force, $F_p = 0.3$. Fasteners attaching the connector to the panel or the structure such as bolts, inserts, welds, dowels, etc., shall be designed to ensure ductile behavior of the connector or shall be designed for 4 times the load determined from F_p." "Fasteners embedded in concrete shall be attached to or hooked around reinforcing steel or otherwise terminated so as to effectively transfer forces to the reinforcing steel." "The value of the coefficient I shall be 1.0 for the entire connector assembly for equation F_p."

The Yugoslavian code, is called the Code of Technical Regulations for the Design and Construction of Buildings in Seismic Regions (Official Gazette of S.F.R. Yugoslavia, No. 31/81). "The seismic forces acting on all elements of a structure shall be calculated according to, $S = K_s K_e G_e$, where K_s is the coefficient of seismic intensity which is 0.025 for Zone VII, 0.050 for Zone VIII, and 0.100 for Zone IX; K_e is 2.5 perpendicular to the flat surface of the wall for partition walls, and non-loadbearing walls; and G_e is the weight of the element of the structure for which the seismic force is being calculated." No information is given for forces acting in-plane for non-loading bearing panels.

For basic information on the codes from which information on cladding panels and connections is included above, the interested reader is referred to a handbook by Paz [1994].

"This unique handbook compiles essential information on the theory, regulation, analysis, and design for the construction of seismically safe structures throughout the world in one comprehensive volume... The focus of the book is on approaches to earthquake engineering from around the world." It contains information on earthquake-resistant design of buildings for the following countries in seismic regions: Algeria, Argentina, Australia, Bulgaria, Canada, Chile, China, Colombia, Costa Rica, Egypt, El Salvador, France, Greece, Hungary, India, Indonesia, Iran, Israel, Italy, Japan, Mexico, New Zealand, Peru, Portugal, Puerto Rico, Romania, Spain, Taiwan, Thailand, Turkey, Union of Soviet Socialist Republics (USSR) which is currently known as the Commonwealth of Independent States [ICS], U.S.A., Venezuela, and (former) Yugoslavia. "Each chapter details a country's geography and geology; history of recent significant earthquakes; and the socioeconomic context of the seismic code and its implementation."

CHAPTER 3

PRECAST CONCRETE CLADDING PANELS AND CONNECTIONS: STRUCTURAL UTILIZATION

3.1 Historical Overview

In another reference by Thiel, *et al.* [1986], the authors included a review of previous work in their description of a feasibility study for a seismic energy absorbing cladding system. For convenience of the reader, the first two paragraphs in their review section are included in this literature survey, along with a summary of the third paragraph:

"The last ten to fifteen years have seen a considerable increase in interest in the effect of cladding on the strength and stiffness of the primary lateral load resisting system. This interest arose with the increase in cost of architectural precast panels, with the increase in cost of the wind bracing system of high-rise building structures and with interest in seismic response of clad systems. Research on seismic effects on cladding is limited. Several analytical studies and a more limited number of experimental investigations are briefly reviewed below. A recent survey Arnold, *et al.* [1987] of architectural detailing practices includes a review of cladding panel practices as well as other non-structural elements. Many investigators have observed the influence of non-structural components (including cladding Goodno, *et al.* [1983]) on the dynamic behavior of buildings. Many measurements of the changes in building period as construction progressed have been made; these observations, in the main, have centered on understanding the dynamic nature of the structural system's response.

"Among the first to promote the idea of using cladding as an integral part of the wind bracing system was Weidlinger [1973]. He observed that cladding can be incorporated into the structural resistance system to increase the lateral stiffness of high-rise buildings and studied the behavior of shear panels. Gjelvik [1973] reported on the interaction between frames and precast panels, focusing on the effect of cladding on the lateral strength capacity of a frame with simple beam-column connections. Oppenheim [1973] was among the first to study the effect of cladding on the dynamic properties of steel building frames. It was concluded that in 'balanced' designs (where panels are of stiffness comparable to the frame) the upper story panels will require large deformation capacities because of the whipping effect. Goodno, *et al.* [1980], Goodno and Palsson [1981], Palsson and Goodno [1982], and Goodno, *et al.* [1983] have investigated the seismic response of clad buildings. It was noted that the addition of cladding stiffness changes the dynamic properties of the structure and causes it to be less or more sensitive depending on the selected ground motion. As a result, it might not always be conservative to neglect the lateral stiffness of cladding during the design process.

"The concept of using the connections between panels to dissipate energy during severe seismic excitations has been investigated in two studies on connections for large panel structures."

Thiel, *et al.* [1986] offered several references on connection design and effects on structural response.

In another reference, Elsesser [1986] presented a survey of seismic structural systems and design implications. He stated, "The search for appropriate structures for optimum seismic resistance has been underway since our building were first damaged in earthquakes. With the development in the past 50 years of dynamic models for buildings, the capability to predict and characterize ground motion in the past 30 years, the recent computational advances, and the recent extensive physical testing of structural assemblies, we are at the threshold of significant developments of unique structures which are truly seismic resistant." He continued, "We have slowly progressed from masonry construction in 1900 to steel and reinforced concrete framing in the 1920s, to welded steel frame assemblies in the 1950s, to ductile concrete frames in the 1970s, to eccentric braced steel frames in the 1980s, and we are now a the beginning of buildings with new concepts using isolation and damping devices. We have also progressed rapidly from static to dynamic systems; those which consider damping, displacement, and energy. These new ideas involve mechanisms requiring tuning and are not conventional brute-force structures."

Elsesser [1986] presented historical and current thinking on seismic demand versus structural capacity, structural systems, isolation systems, and architectural implications. He noted, "In contrast to the conventional structures, the new concepts attempt to provide for seismic energy dissipation in a controlled manner by distortion of a distinct element or series of elements. We project the following basic behavior with damping cladding: Moderate lateral story drifts required to satisfy the energy demand. Response reduced and limited by high damping mountings of the cladding."

For damping cladding for steel moment frames, Elsesser [1986] noted that the optimum location is along the perimeter of the building. The expected earthquake damage is "reduced interstory drift" and "low to moderate non-structural damage." Elsesser stated that the impact on architectural design would include the following: cladding design must be coordinated with the damping system, occasional access to dampers is desirable, and the configuration is important.

Goodno and Craig [1989] presented an historical overview of studies on the contribution of cladding to lateral resistance of buildings. The abstract of their paper is as follows: "The present paper will review research that has been carried out over the past two decades to examine the role that architectural precast cladding systems can play in providing lateral resistance to building structures. Architectural precast is often dismissed as nonstructural and does not normally form part of either precast frame or wall-panel building construction. Nevertheless, studies carried out by several groups over the past few years have amply demonstrated that such architectural precast systems do indeed contribute measurably to the lateral resistance of buildings. The paper reviews the early analytical studies and initial full scale testing that were carried out to determine the scope of the problem. Next, the more recent efforts of a number of groups to analytically model the details of precast cladding systems on buildings using materials and structural response data from laboratory tests are discussed. A number of these studies were motivated by the need to understand how such structures will respond to earthquakes. Finally the review concludes with observations on promising directions for continuing and future research in this increasingly important area. The paper also includes a comprehensive chronological bibliography of relevant research carried out over the past two decades." The bibliography contains 78 references, about one quarter of which were published before 1980. [Note: Pre-1984 references are recorded on paper in the Abstract Journal that was first published in 1971. The first year of extensive coverage for the online data bases is 1984.] Most of the post-1980 publications are included in this literature survey.

3.2 Cladding: Levels of Contribution to Seismic Resistance

According to Arnold [1989], "For the possible contribution of cladding to the seismic resistance of a building, four levels of participation can be identified:

1. "Theoretical Detachment: This is represented by the typical push-pull detail for cladding attachment used in California. While, in theory, the ductile rod connection detaches the cladding from the structure, in a building with hundreds of cladding panels it is likely that the detachment is not complete, and there is some transmission of forces from the structure to the panels and vice versa.
2. "Accidental Participation: This occurs with connections such as slotted connections and sliding joints in which, because of being or errors in installation, the separation between the cladding and structure in not effective. The is uncontrolled participation.
3. "Controlled Stiffening or Damping: This involves the use of devices to connect the cladding to the structure in such a way that the damping of the structure is modified (usually increased) or the structure is stiffened.
4. "Full Structural Participation: The cladding and the structure become a new integrated composite structure in which each element performs an assigned tole. The cladding may participate in vertical support, and definitely contributes to lateral resistance."

"In theory the fourth level of participation makes the most economic and dynamic sense because the cladding is removed from its role of dead weight to one of integral support: in practice this level has proved difficult to achieve, and it has proved more economic (if not more performance effective) to use level one. Study of other structures in the dynamic environment, such as airplanes and automobiles, has shown a steady evolution from level one to level four. Today's building cladding compares to the doped fabric of a 1920s wood-structured airframe."

Iverson [1989] commented that "there are many exciting and encouraging ideas that apparently have found their time."

"Seismic Isolation: This principal is rather straightforward in the basic application and in finding considerable acceptance in seismic regions throughout the world. The entire structure is set on large elastomeric bearings which are quite stiff in a vertical direction and flexible in the horizontal direction. When the earthquake loads are applied, the movements the structure experiences are attenuated by the isolators and the loads the structure experiences are much reduced. This then reduces the seismic loads on the cladding and its connections.

"A secondary concept then is to use a similar system with mini-isolators to connect the panels to the structure. There are several obvious problems that must be evaluated, but concept and physical research continue and developments will follow,

"One point that lends interest to all of these efforts is the relative cost that the connections contribute to the final in-place, overall cost of the panel. A major part of the casting and design effort and much of the erection and final positioning of the panel are related to the connection. The material and labor involved in the item are only a small part of its major overall effect on cladding costs.

"Damping: The largest potentially beneficial unknown in the dynamic equation is damping. Various schemes have been proposed in recent times to maximize damping in structures. [A figure presented in the text] illustrates the decrease in seismic force with increased damping. Reduction of motion and forces and hence cost of the structure follows. Logic then is to incorporate the damping into a necessary part of the structure and connections of the cladding seems a logical place. Again considerable development and testing will be necessary, both to establish the viability of the scheme and to validate its damping effect. Clear measurement of damping in existing buildings still remains controversial, or at least an item of disagreement in the profession.

"Load Bearing Cladding: Finally, utilization of the cladding in the vertical and horizontal load resisting system should be considered... The primary weakness that must be overcome is the old problem of shrinkage..."

3.3 Architectural Implications for Structural Cladding

Arnold [1989] noted, "If the cladding models and architectural trends... are evaluated for their implications on the use of cladding for resisting lateral forces, some conclusions are immediately apparent."

Arnold continued, "Of the three classical models of facade arrangement, the vertical pattern cannot help in the provision of lateral resistance, but it might form the basis of a composite structure for resisting vertical forces. The horizontal or spandrel form cannot help in bracing between floors, and to the extent that the spandrel becomes stronger or stiffer it can lead to a detrimental strong beam / weak column situation. The rectangular form, using a panel that spans at least from one floor to the next, with a window size sufficiently small to permit useful shear strength in the panel remains as one of the useful patterns. ...This form, so long discredited, has returned as a part of the drive towards more traditional forms.

"Obvious problems are presented by cladding patterns in which models are mixed. The more arbitrary and irregular the mix the greater becomes the difficulty of developing a rational structurally participating cladding system. And, clearly, the greater the general irregularity of the building, the more desirable it would be to limit the participation of the cladding in order to avoid the possible amplification of stress concentration or torsional effects."

3.4 Conditions for Effective Structural Cladding

Arnold [1989] also commented, that "Although, on the face of it, architectural trends do not seem to be propitious for the promotion of structural architectural cladding, if structural rationality could lead to significant performance and cost benefits, then the architectural discipline necessary will begin to be requested by building owners. Notwithstanding the strong architectural trends there will always remain some owners who are seeking a functional, cost-effective, and simple building."

Before considering issues of responsibility, the designer needs to clearly state what he means by "structural cladding." That is, he must identify the vertical plane in which the panels will be located, and potential advantages and problems with the design. Typically, structural cladding panels are located in the vertical plane just outside of the vertical plane of the perimeter framing.

3.5 Issues of Responsibility

Stockbridge [1990], a forensic engineer, outlined lessons learned from cladding failures. From his experiences with precast concrete cladding used in the U.S., he noted, "Problems in precast panels often result from an unclear distribution of responsibility between the design engineer of records and the panel precaster engineer."

Stockbridge continued by pointing out general lessons such as: "Whether the reinforcing and connections are designed by one or the other is optional, but it must be clearly defined. If the code permits, and the precaster has demonstrated design experience, there can be advantages in delegating the detailing to him. An experienced precaster is in a position to select the details that are best suited to his specific production and erection procedures. If the precaster's engineer does the detailing, he should be registered and seal the shop drawings.

"For economy, sometimes the same concrete mix is not used [for the] full thickness of the panels. In cases where these mixes have not been reasonably compatible, bowing and warping have occurred. As a minimum, ...the water/cement and cement/aggregate ratios should be similar. Panels with a normal-weight concrete face mix and a lightweight concrete back-up mix should be selected with great care.

"Some prefabricated concrete panels incorporate insulation. In such panels where there has been significant lateral restraint between the inner and outer layers, bowing has sometimes occurred."

Spronken [1989], a precast concrete cladding fabricator, discussed issues of responsibility for structural cladding and structural framing, and raised some questions about the "consideration of the behavior exhibited by the superstructure relative to the composite behavior of the precast concrete wall cladding system in resisting horizontal loading."

He noted that "If cladding is to be used to partially or wholly contribute to the lateral resistance of a building it must be designed by and be the responsibility of the Structural Engineer of Record. The cladding will now form an integral part of the structure rather than be an appendage to it. It does not appear reasonable to simply revise the specifications which will make the precast concrete contractor, who is now only responsible for the cladding, responsible for the behavior of the structure as well."

He continued, "Is not within the scope of work of the expertise of the precaster to know intimately the behavior of a particular structure unless the precaster is fully appraised of all of the criteria pertaining to the behavior of the structure. This would include tributary loads, deflections, creep, etc. This information can only be provided by the Structural Engineer. If this is the only result of this trend, we would welcome such a development as it has been our opinion for many years that the cladding was often relegated to the status of decoration rather than a structural element. In fact, the precaster and his engineer who were often unaware of the overall building behavior and consequently the cladding often contributed involuntarily to the lateral restraint of the structure resulting in less than satisfactory results... If the Engineer of Record is not responsible for this aspect of the work then the entire subject should properly be ignored and returned to the premise that whatever resistance is to be offered will be contributed by the caulking through shear and tension. Value unknown but more than zero."

"Let us assume for the sake of this discussion that the decision has been made to mobilize the exterior cladding to provide some lateral resistance to the structure, then the following information must be available:

1. "What percentage of the load must be taken by the exterior cladding? What is the nature and direction of these loads?
2. Will the cladding behave with the same deflection characteristics as the building frame in:
    a. "bending mode? or
    b. "shear mode? or
    c. "combination of the above?
3. "What are the deflection characteristics of the structure under combined action?
4. "What thermal movements must be accommodated between the building frame and the cladding while maintaining the structural integrity required?
5. "What are the long term creep characteristics of the building frame relative to the cladding?
6. "What other deflections may be present with the building frame which will impact upon the cladding?
7. "What are the limits of tolerance which be accepted without revisions being made to the connections and what method of repairs must be undertaken if these limits are exceeded?"

He continued, "It is obvious if the cladding is to provide lateral resistance it must have connections which will permit these loads to brought to the base of the building in a determinate and straight-forward manner. This must be identified by the Structural Engineer with the same or greater clarity used in setting out the steel or concrete frame criteria and design. Because the cladding panels incorporate Architectural elements, these loads may well require greater attention than is presently being paid to the purely structural elements of a building. In particular, the matter of deformations must be addressed. Certainly there will be requirements of more extensive consultations with the Architect to achieve a workable solution."

Spronken also presented some concepts on how to attach cladding panels. He concluded by stating, "Coordination (among) the various disciplines will be increased since this element will transcend the Structural and Architectural aspects. Much work remains to be done in what should prove to be the next step."

Hegle [1989] reviewed special considerations that are required for structural cladding and connections. He stated, "Precast cladding panels and their connections which are required to participate with the structure to transfer lateral wind and seismic loads must have some special characteristics such as the ability to:

1. "Carry loads after distortion and yielding due to building movement.
2. "Support load reversals without failure.
3. "Develop strength and ductility."

He continued, "The cost of structural cladding will surely be greater but may be partially offset by reduced structure cost and less architectural damage due to reduced story drift under seismic loading."

CHAPTER 4

RESEARCH ON PRECAST CONCRETE CLADDING PANELS AND CONNECTIONS: STRUCTURAL UTILIZATION

Research on the structural utilization of precast concrete cladding panels and connections can have experimental and/or analytical components.

For the experimental components, there are no standards for structural cladding, let alone for cladding as currently design, that is, isolated from participation in the lateral load resistance of the structural framing. Stockbridge [1992] commented that "While there are numerous design guidelines [for the isolation of cladding], there are no standard methods for testing cladding systems for seismic performance. There forces and distortions that a building system experiences during an earthquake depend very strongly on its dynamic behavior, that is, not only on it natural period and damping but also on the manner and extent to which it yields... The requirements of the seismic provisions (for isolated cladding) can usually be substantiated by means of calculations and illustrative details on the construction drawings. When conformance to the seismic provisions cannot be thus substantiated, testing may be required... Normally, it is sufficient and more economical to mock up and test critical elements of the cladding assembly. However, there may be cases where whole assemblies consisting of one of several precast elements must be tested... A properly developed mock-up should include in-place racking distortions and out-of-place distortions representing, at a minimum, interstory drifts three to four and one-half times the calculated displacements caused by the required [design] seismic forces. The tests should include cyclic displacements to simulate earthquake-caused motion. The results of the tests should be able to show that the connections will permit the required movement... In addition, if sufficient clearance between adjacent elements is not maintained, tests should demonstrate the ability to withstand the effects of interaction between the elements without creating a hazard."

Development of Code Requirements: The Energy Dissipation Working Group of the Base Isolation Subcommittee of the Structural Engineers Association of Northern California has been developing code requirements for the design and implementation of passive energy dissipation systems. This development included the consideration of structural cladding with energy-dissipating cladding connections, as well as the consideration of the usage of energy-dissipating devices within the structural framing of the superstructure.

In a 1993 paper, Whittaker, *at al* prepared a paper on the document to date. Since 1993, the document has undergone review and revision, and may be modified further by needs identified after the 17 January 1994 Northridge earthquake.

The abstract of the paper is as follows: "Passive energy dissipation (or supplemental damping) systems have been used for seismic applications in buildingd in Canada, Italy, Japan, Mexico, New Zealand, and the U.S.A. The implementation of passive energy dissipators, and the considerable research effort in the United States over the past five years have resulted in a need to develop requirements for the design and implementation of passive energy dissipation systems."

"The Energy Dissipation Working Group (EDWG) of the Base Isolation Subcommittee of the Structural Engineers Association of Northern California (SEAONC) has developed a document entitled 'Tentative General Requirements for the Design and Construction of Structures Incorporating Discrete Passive Energy Dissipation Devices" (hereafter known as the *document*). The EDWG intends that the *document* supplement the *Uniform Building Code (UBC)* with additional design requirements developed specifically for building incorporating supplemental damping devices. The *document* provides general design requirements applicable to a wide range of system hardware to confirm the engineering parameters used in the design process. The paper describes the analysis, design and testing requirements mandated by the *document* and the rationale behind its development."

Whittaker, *et al.,* continued, "The general philosophy of the *document* is to confine inelastic activity, in a structure incorporating passive energy dissipators (PEDs), primarily to the energy dissipators and for the gravity load-resisting system to remain elastic for the Design Basis Earthquake (DBE). Since the dissipators do not form part of the gravity load-resisting system, they are replaceable after an earthquake and as such this type of innovative structural system is fundamentally different from a conventional seismic lateral load resisting system." [It is intended that the energy-dissipating cladding-to-frame connections do not carry gravity loads.]

"A hierarchical nomenclature is used in the *document.* Passive energy dissipation devices are known an energy dissipation units (EDUs). EDUs form an integral part of an energy dissipation assembly (EDA); the EDA is a one-bay, one-story assembly composed of the EDUs and the elements that provide lateral and vertical stability to the EDUs... The energy dissipation system (EDS) is the three-dimensional collection of all of the EDAs.

"The *document* provides general design requirements applicable to a wide range of possible systems. In remaining general, the *document* relies on testing of system hardware to confirm the engineering parameters used in the design, and to verify the overall adequacy of the EDUs and the EDS. In general, acceptable systems will: (1) remain stable for required design displacements; (2) provide non-decreasing resistance with increasing displacement (for rate-dependent systems); (3) not degrade under repeated cyclic load at the design displacement; and (4) have quantifiable engineering parameters (e.g., force-deflection and energy dissipation characteristics).

"There are two types of energy dissipation devices recognized in the *document:* rate-dependent and rate-independent devices... The only rate-dependent devices explicitly recognized in the *document* are viscous and viscoelastic PED devices. The rate-independent PED devices implicitly recognized in the document are: friction-slip, steel-yielding, and shape-memory alloys.

"The *document* prescribes the use of dynamic analysis procedures to determine maximum responses. Dynamic analysis procedures include both response spectrum analysis and linear and nonlinear time history analysis. Linear procedures can be used for the earthquake resistant design of structures incorporating viscous or viscoelastic energy dissipators. Nonlinear time history analysis is mandatory for non-compliant rate-dependent EDSs and for all rate-independent EDSs.

"The seismic demands are described by the spectral demands of the DBE. These spectral demands correspond to a level of ground motion that has a 10 percent probability of being exceeded in a 50 year time period. For building design not using a site-specific hazard analysis, the design -basis spectra are defined by the ground motion spectra specified by the *UBC* for dynamic analysis of conventional buildings. The seismic design actions and deformations in the EDS are based on the DBE analysis. Stability of the EDUs must be verified by test for the displacements corresponding to the maximum level of earthquake ground motion that may be expected at the site.

"The *UBC* places a lower bound on the design actions and deformation computed using dynamic analysis. Similarly, minimum base shear coefficients at the ultimate (strength) level are specified for the EDSs. The minimum base shear coefficient is calculated as ZC/R_W using the method specified in the *UBC,* and scaled to the ultimate (strength) level, via a material-dependent conversion factor, for comparison with the results of the dynamic analysis. The minimum base shear coefficient is dependent on the type of lateral load resisting system: for EDSs with no supplemental moment frame ·non-dual system), the minimum base shear coefficient is computed using an R_W of 10; and for dual system EDSs, the minimum base shear coefficient is computed using an R_W of 12.

"The *document* was prepared in keeping with the most current information and the present state-of-the-practice of energy dissipation. However, seismic energy dissipation is a relatively new technology and there are many design-related issues that require additional research. In establishing the design requirements in the *document,* the EDWG recognized these limitations and chose a conservative approach in developing design requirements. As experience with energy dissipation systems increases, and as the results of related research becomes available, the design requirements will invariably be refined."

The interested reader is referred to the paper for the tentative general requirements.

4.1 RESEARCH GROUP: Collaboration of San Francisco - Bay Area, California, practitioners and professors.

Reference: Thiel, *et al.* [1986]. "Seismic Energy Absorbing Cladding System: A Feasibility Study,"

Type of Study: Analytical

Abstract: "A new approach to provide added damping through controlled activation of part of the lateral force resistance capacity of cladding panels is presented and shown to be feasible. Added damping is obtained through inelastic behavior or friction of specially detailed connection devices between the structure and the cladding panels (hysteretic damping). A nonlinear computer analysis indicates that substantial damping can be introduced into a structure's response through suitable modifications of the connection details. It suggests that a steel ductile moment frame with 2% critical damping may have its effective viscous damping increased to 8% or more and resulting base shear and peak roof displacement reduced by 50%. This approach to increasing the damping has the advantages that: (1) energy can be dissipated mechanically over the building's height rather than by local inelastic action of a limited number of structural members; (2) the induced inertial loads during an earthquake can be reduced thereby protecting the structural system; (3) there is no yielding of materials in the load bearing system in the process of energy dissipating, thus the capacity of the building to withstand additional earthquakes is not compromised; (4) the amplitude of vibration can be considerably reduced, thus increasing damage control for interior contents and exterior finishes, which constitute more than 50% of the value of the building; and (5) increasing safety for the structure's occupants."

Attachment Devices and Configurations: "There are two basic strategies for attachment of the panels to provide panel participation in structural response: (1) through adding an attachment element, in addition to the regular support fixtures, that provides only dissipation; and (2) through modifying the basic fixtures to provide gravitational support and energy dissipation. The latter may have advantages for new installations, while the former is attractive for rehabilitation applications.

"Among the several possible basic mechanisms to provide dissipation, three seem most attractive for investigation: (1) friction; (2) yielding of a metal; and (3) viscous behavior of a polymer. The first two are displacement sensitive, while the latter is velocity sensitive." The paper contains information on these devices as used or developed by others.

Analytical Studies: Technical feasibility is discussed including panel force capacity, theoretical equivalent damping, building response incorporating damping from cladding, and attachment devices and configurations.

For panel force capacity, the authors noted that "The premise of this proposal is that a significant amount of damping can be introduced into the structure's response through activating the shear capacity of the panel and by taking advantage of the hysteretic behavior of their connections. The first question in determining feasibility is what magnitude of shear forces typical (precast concrete) cladding panels can withstand. The shear capacity of a panel is limited by the capacity of the vertical pier section." The paper contains a table that "reviews the shear capacity, maximum displacement, and the tie-down force for dimensions that are at the extreme of typical practice and are therefore conservative."

For theoretical equivalent damping, the authors discussed the theoretical equivalence of viscous damping for a single degree-of-freedom system and its approximation of damping in a real structure.

Assumptions for Analysis: "Technical feasibility now rests on whether elasto-plastic attachment devices with yield strength less than the panels' capacity are sufficient to add appreciable damping to the structure, since typical cladding panels have the capacity to resist substantial shear forces. For this purpose, a hypothetical 15 story building of uniform mass and stiffness (for computational simplicity), and four 20' bays. The dampers attaching the panels to the structure are lumped at each floor and idealized as having elastic-perfectly plastic behavior. The mass of the panels is incorporated into the overall mass of the building, assumed to be 100 psf."

Software: DRAIN-2D

Analyses: nonlinear time-history analysis.

Ground Motion: a base motion accelerogram consistent with the ATC-3 0.4g spectrum (S1).

Response Quantities: roof displacement and equivalent base shear for different viscous damping ratios of the structure system and for different attachment characteristics.

"The response quantities suggest:

1. "Within the range examined here, the effectiveness of the device increases with increasing yield level.
2. "The device requires relatively high stiffness, comparable to the structure's story stiffness, to be most effective.
3. "For the high yield levels and 2% viscous damping in the frame, the damper reduces the response of the structure by approximately 40% as measured by maximum roof displacement and even more as measured by roof acceleration (50%) and base shears (45%).
4. "The devices are especially effective in damping out higher mode response; thus, the benefits gained from reductions in maximum floor accelerations and therefore damage to components and contents could exceed the benefits from reduced from forces."

Summary: "This paper has shown that the effective damping of a building can be substantially increased through activation of part of the lateral force resistance capacity of cladding panels and controlled hysteretic behavior of their connections to the structure. Typical panel configurations have the force capacity and attachment fixtures can be designed to provide substantial damping. In this way:

1. "Energy can be dissipated mechanically throughout the height of the building rather than by localized inelastic action of main structural members.
2. "The induced inertial loads during the earthquake can be reduced thereby protecting the structural system.
3. "There is little yielding of materials in the load bearing system during energy dissipation, thus the capability of the building to withstand additional earthquakes is not compromised.

4. "The amplitude of vibration can be considerably reduced, thus reducing damage to non-structural systems, which constitute more than 50% of the value of the building.

"This study has demonstrated that it is technically feasible to add substantial, beneficial damping to a building's earthquake response through activating the cladding's shear capacity with appropriately designed hysteretic attachments. Many questions remain concerning the attachment fixtures detailed design and performance, structural, panel and attachment performance under service loads, fireproofing, alterations in manufacture and installation, and cost-effectiveness that need to be answered before this system is appropriate for application. The answers to these questions will have major bearing on the economic and technical practicability of using the proposed system, however, they will not affect technical feasibility."

4.2 RESEARCH GROUP: Depts. of Civil Engrg., Univ. of New Hampshire, Durham, NH, and Univ. of PA, Philadelphia, PA.

Reference: Henry and Roll [1986]. "Cladding-Frame Interaction."

Type of Study: Analytical.

Abstract: "Analysis of the interaction between a precast wall cladding panel and a reinforced concrete frame under linear static and dynamic forces has been performed. Two computer programs (LDYN and LSTAT) were written to evaluate the effects on the lateral displacements and dynamic characteristics of a moment resisting frame when the cladding is incorporated into the analysis as a structural component. The data indicate a significant change in lateral displacement, natural frequency, member force distribution and connector forces when compared with a frame neglecting the structural characteristics of the cladding."

Design Philosophy: "The alternative [philosophy] is to design an integrated system which would incorporate the lateral stiffness and damping characteristics of the exterior cladding in the seismic analysis. This would produce a more efficient and cost-effective design. These elements could be designed to resist moderate seismic movements with little or no damage. Under severe seismic loads the elements would help dissipate the energy generated. The predicament arises from the fact that there presently exists sufficient information to design by [eliminating the interaction], but insufficient engineering knowledge for[(incorporating the structural effects]."

Analytical Studies: The authors described the development of the analytical model used for their cladding element, including the development of the stiffness matrices for the connectors.

Objectives: "The purpose of this research was to study the behavior of the cladding-frame interaction for reinforced concrete structures by means of computer models. Each cladding panel is attached to the frame at only four distinct locations [at the vertical panel edges near the corners, appearing attached in the plane of the structural frame]... The goal was to provide an engineer with some additional information so that a judicious engineering decision might be made on how best to utilize the interaction in a particular design project. The research was conducted on an analytical level as the first step toward a more comprehensive understanding of the interaction process."

Assumptions: "Initially two models were investigated: A finite element model and an equivalent cross bracing model. The equivalent cross bracing technique was extremely attractive due to its ease of application. Unfortunately, the simplified nature of the model limited the extent of the applicability with respect to the problem under investigation."

Description of Analytical Models:

Software and Analysis Types: "Two computer programs were developed to simulate the cladding-frame interaction under the following analytical situations: (1) Linear elastic statics (LSTAT); and (2) linear dynamics (LDYN)."

Major Results: "It was found that there are three major conclusions which could be drawn from the data gathered. The first involves the assumption used in the analysis of a building system which utilizes precast wall cladding as an exterior enclosure for a moment-resistant, reinforced concrete frame. Presently, the structural aspects of the exterior cladding are neglected during the analysis; this procedure is presumed to be conservative. Unfortunately, this is not always the case. If the connectors are designed to eliminate cladding-frame interaction, then the assumption will remain valid as long as there is room for the connectors to slide and/or rotate. Once the movement equals the gap available, the connector becomes restrained and the... natural period of vibration will change significantly. Thus when an equivalent lateral static force procedure is used, the applied loads calculated when the panel is included would be greater than the loads calculated when the panels are neglected.

"This could also happen if the connectors are not installed properly. Besides, all the potential benefits of the cladding-frame interaction would be wasted. These benefits include: Smaller relative lateral displacements, smaller member forces for the beams, and reduced moments in the columns of the lower stories. As a result of the lesser member forces, smaller member dimensions would be possible, and thus a reduction in construction costs.

"The second conclusion concerns the design of the connectors which are usually analyzed to handle just the gravity load of the cladding elements. ...the forces acting on the connectors can be quite large and should be known in order to provide an adequate connector...

"The third conclusion involves how the structural response of the system is affected by the type of connector used. It was anticipated that using all fixed connections or all simple connections would produce results which were significantly different. As it turned out this was not what happened. Upon further thought on the matter, the reason for this response became clear. The system under investigation had the external applied loads acting parallel to the longitudinal axes of the cladding elements and the connectors. In addition, the vertical displacements are small in comparison to the lateral displacements. In fact, the axial degrees of freedom are often neglected during an analysis of a shear type building such as the one modeled for this study."

4.3 RESEARCH GROUP: Dept. of Civil Engrg., Univ. of New Hampshire

References: Henry, Goodspeed, and Calvin [1989]. "A Simplified Box-Frame Model for Structural Cladding Panels."

Type of Study: Analytical.

Abstract: "The purpose of this paper is to develop a simple mathematical box frame model consisting of beam elements to represent architectural precast concrete cladding (APCC) as a structure member. The model simulates the behavior of a structural cladding panels in a plane frame analysis with sufficient accuracy to be adequate for design. The forces, lateral drift and panel stresses determined by the model in a two dimensional frame analysis compared favorably with the results of a finite element analysis. The new model enables an engineer to confidently include APCC as a structural lateral load resisting building component. An approach is presented that used the box frame model and relationships between the box frame model forces and finite element panel stresses. An example is presented to illustrate the use of the new model."

Analytical Studies:

Objectives: "The main objective in developing the box frame model was to establish a beam model that would accurately determine the in-plane displacements of the structural frame and the stress distribution in the panels under lateral loads. The box frame model has the ability to develop flexural as well as shearing strains."

Assumptions: The panels appear to be connected to the columns in the same manner as was done by Henry and Roll [1986].

Description of Analytical Models: "The box-frame model... was developed to represent a precast cladding panel connected to the building frame at the four panel corners... The box frame model is composed of four box-like frames, called panel-boxes... The four identical panel-boxes are attached at adjacent corners to represent a single APCC panel.

"Each panel-box is composed of four beam elements that are rigidly connected. The panel box horizontal beams are used primarily to model the panel flexural characteristics and the vertical beams model the panel shear characteristics." For further details, please refer to the cited paper.

Software: PAFEC (for static analysis)

Conclusions: "The box frame model presented in this paper meets the analysis need for treating APCC panels as structural components. The comparison of the macroscopic structural responses predicted by the box frame model are within the design limits of the responses obtained by a finite element mesh.

"It is important to emphasize the limitations of the model. ...the model provides acceptable analysis results for APCC used as a structural components in the range of height to length ratios of 0.15 to 0.6. Although panels with height to length ratios greater than 0.6 were not considered in this investigation, it is felt that the box frame model can be successfully used with ratios as high as 2.0. The forces induced in the mid-height beam were to have a poor correlation with the forces produced with the finite element model; designs should be based on floor loads for these beams.

"Increasing the number of box frame elements used to model an APCC panels does not result in a better correlation with a finite element model; [in] fact the correlation becomes extremely poor. The addition of the extra pin joints drastically reduces the stiffness of the box frame model.

"This investigation did not include the application of forces applied to the panel. Therefore, no conclusions are made on the accuracy of the results obtained with the box frame model when the APCC is subjected to a direct load."

4.4 RESEARCH GROUP: J.R. Harris & Company, Denver, Colorado

References: Charney and Harris [1989]. "The Effect of Architectural Precast Concrete Cladding on the Lateral Response of Multistory Buildings."

Type of Study: Analytical.

Abstract: "Architectural precast concrete cladding can have a positive or negative effect on the response of buildings to lateral loads. The effects are generally positive if the cladding and its connection to the superstructure is consistently and rationally included in the analysis and design. Cladding influence can be negative, and in some cases disastrous, if the presence of the cladding unintentionally stiffens the structure or restrains the free deformation of certain elements.

"In this paper, the role of cladding in the lateral load resistant design of buildings is evaluated. Since existing knowledge on the behavior of cladding and its connections is insufficient to develop a rational design methodology for any limit state or loading, recommendations for coordinated analytical-experimental research are forwarded."

Analytical Studies: "[Here] the effect of architectural precast cladding, whether advantageous or adverse, is investigated from the point of view of limit state design. For wind and earthquake, the basic limit states are serviceability, strength, and stability. For earthquake, the additional consideration of energy dissipation capacity is critically important because the safety limit states are defined to include nonlinear inelastic behavior. Each of the limit states is discussed in some detail. Where possible, previously existing research is drawn upon to support points being made. In other cases, the results of new calculations on building-cladding response are presented."

Under serviceability limit state design, the authors discussed drift, methods of structural analysis, influence of cladding on stiffness, uncertainties in modeling cladding-connection behavior, finite element analysis of laterally connected panels, impracticality of finite element approach, P-delta effects, and effect of cladding on accelerations of upper stories. The authors also discussed cladding strength and energy dissipation requirements.

Software: SAP90 Finite Element Analysis Program, DISPAR Post-Processor for SAP90.

Summary and Conclusions, and Needed Research: "The role of architectural cladding on building response has been addressed from the limit state design point of view. As is apparent from the discussion, the primary conclusion that can be drawn is that adequate information does not exist to form a rational design basis. Hence, efforts should be made to conduct coordinated experimental-analytical research, the objective of which would be to develop rational analysis and design techniques which are applicable to all limit states. The scope of such research should include, but not necessarily be limited to the following:

1. "Perform extensive field tests of existing buildings which use architectural precast panels as faces. Determine, on the basis of forced vibration tests and on response measured during moderate environmental excitation, the effect of cladding on response. These effects should includes stiffness (frequency) and damping. Data obtained from buildings instrumented by the USGS program (and others) could be used to evaluate response at moderate levels of seismic excitation.

2. "On the basis of information obtained in (step) 1, correlate response with building type, cladding characteristics, and connection detail.
3. "Perform tests on free and rigid cladding connections to determine the load-deformation response, damping characteristics, and energy dissipation capacity of the connections. Some work along these lines has already been accomplished for connections typically used in precast panel construction (see paper for references). New tests should include panel elements supported to a mock up of the superstructure. Tests should be performed both statically and dynamically, and at a broad range of frequency and amplitudes of response. Panels and their connections which has been exposed to weathering should also be tested.
4. "On the basis of information obtained in (step) 3, develop analytical force-deformation response relationships for levels of loading through slipping, yielding, and failure of the connection material.
5. "Develop three degree-of-freedom linear and nonlinear spring element for analysis which emulate the response of connections. Attempt to analytically correlate the response of cladding-superstructure subassemblages tested in (step) 3.
6. "Determine either analytically or experimentally if the flexibility of the panel element can be ignored when predicting response.
7. "Alter one or more currently existing computer programs to accept the 3-DOF panel connector element developed in (step) 5, or to accept the panel-spring element (as illustrated in Fig. 6 in the paper).
8. "Using the modified computer program, attempt to correlate the analytical response with the experimental response of structures such as those investigated in (step) 1 above.
9. "Develop analysis and design recommendations."

4.5 RESEARCH GROUP: School of Civil Engrg. and Engrg. Sci., Univ. of Oklahoma, Norman, OK.

References: Sack, Beers, and Thomas [1989]. "Seismic Behavior of Architectural Precast Concrete Cladding."

Type of Study: Experimental and Analytical.

Abstract: "The nature of architectural precast concrete cladding (APCC) participation in the structural response of high-rise buildings subject to seismic excitation is explored using full-scale laboratory studies and companion analytical investigations. Four basic insert types, plus various combinations of connector bodies, were tested experimentally to obtain the static stiffness properties and a limited amount of low cycle fatigue data. Full-scale tests were run on a one-story single bay structural assemblage, which consisted of a steel rigid single bay with two precast concrete panels attached with dissociative connections at the top and integrative connections at the bottom. The assemblage was subjected to earthquake loading through a closed loop servo-controlled hydraulic loading system. Exceedance levels, plus power spectra density and time-response plots, were obtained for twelve transducers. The experimental results were augmented with static and dynamic finite element analysis of the assemblage. The results from the experimental phases of the work were utilized in a feedback loop to the analytical studies."

Overview of Study: "The study reported herein was limited to the most basic models of connections. These basic types consisted of ferrule inserts with threaded rods, and standard angles with an appropriate fastening mechanism (e.g., welded inserts only, and inserts with face plates). Most connections are more complex than these basic components. Full scale connection assemblies mounted in reinforced concrete block, measuring two feet on each side and from six to eight inches thick, were tested to obtain the load-displacement characteristics and energy dissipation capabilities of the various connections. These mechanical properties were compared to those values predicted by classical structural mechanics methods, and finite element analysis. Subsequently, a one-story, single-bay full scale assemblage containing two precast panels were subjected to earthquake loading. We also analyzed the assemblage using the finite element method by incorporating the data from the connector study."

Experimental Program: For a detailed description of the reaction frame and the two side-by-side full-scale (6' x 12' x 4") specimens, the interested reader is referred to the paper.

Type of Loading: Static tests to determine the flexibility coefficients. Sinusoidal displacement-controlled input through a range of frequencies to determine natural frequencies. Displacement-controlled time history from 1971 San Fernando earthquake record of 9th floor of the Jet Propulsion Lab (chosen because it exhibited high amplitudes in the absolute acceleration response spectrum).

Main Findings (narrative and graphics)

Analytical Studies: "The purpose of the analytical study was to predict the interaction between structural framing and precast curtain walls. The method of analysis used was based on the displacement method. The structural framing was idealized as beam, truss, and spring elements and the precast curtain walls were idealized as two-dimensional panel elements."

Software: SAPFAP, SAPIV.

Analysis Types: "Linear static analysis. Nonlinear static analysis assuming the structural frame behaves linearly elastic and the panel-frame connections behave with material non-linearity. Dynamic linear analysis based on the methods used in SAP IV."

Conclusions: "The connector studies demonstrate that panel connections perform as ideally elastic perfectly-plastic materials. Face plates do not enhance connections incorporating single inserts and threaded bars. Anchor plates using Nelson studs or welded rebar for attachment are acceptable for attaching support angles and lateral connections. During our cyclic tests, the concrete of the panels maintained its integrity. The energy dissipation characteristics for a connection systems can be based on the product of the interstory drift and the plastic load limit. From our analytical studies we found that panel configurations using integrative connections provide additional lateral stiffness to the system while loaded in the linear elastic range but will supply negligible additional stiffness if loaded in to the post–yield range. Panel configurations using the dissociative connection systems provide negligible additional lateral stiffness. The full-scale assemblage performed well when subjected to a recorded earthquake.

"Our analytical results show that the full-scale test assemblage with cladding has 17% greater lateral stiffness than the bare frame. While the experimental measurements of stiffness do not corroborate this fact we are confident of the analytical results. The panels increase lateral stiffness in the elastic range. The first three analytically predicted natural frequencies of the full–scale assemblage agree reasonably well with measured values.

"The overall performance of the APCC and its connections was excellent. We believe that high-rise buildings that make use of the APCC technology will respond satisfactorily during a severe seismic event. Panels may sustain superficial damage, but should remain on the building and intact."

There are no conclusions or comments about the use of two panels per bay in the study in comparison with the more commonly used full-bay panel.

4.6 RESEARCH GROUP: PRESSS, c/o Englekirk and Sabol, Los Angeles, California.

References: • Englekirk, R.E. [1989]. "Towards a More Effective Use of Precast Concrete Cladding." • Nakaki, S.D.; and Englekirk, R.E. [1991]. "PRESSS Industry Seismic Workshops: Concept Development."

Background Information: Englekirk [1989] examined "issues that [affect] the utilization of precast concrete panels as a cladding element in regions where the bracing system must be capable of sustaining seismic action. When the facade of a cast-in-place concrete building becomes part of the bracing system it must conform to requirements for the design of ductile moment resisting space frames. These requirements include: (1) a maximum span to depth ratio (to ensure flexural ductility) of 4; (2) a minimum aspect ratio ($b_{min} \geq 0.3d$); (3) closed hoop stirrups are required in the hinge region; and (4) column strength must exceed beam strength.

"...Clearly the bracing concept is not well-suited to precast concrete cladding. If precast concrete cladding is to be effectively used to brace a building its cost effectiveness requires that: (1) cladding panels be light and contain substantially less volume of concrete; (2) connection details must recognize the fact that the interface will be between two precast elements or between a precast element and cast-in-place column [with the panels typically being placed in the same vertical plane as the exterior structural framing]; and (3) requirements for concentration of reinforcing, especially closely spaced hoops, must be mitigated if panel quality is to be assured and panel sizes minimized. The satisfaction of these objectives requires design innovation and technology development."

Englekirk continued, "...Key to attaining an effective seismic bracing system constructed using precast concrete cladding is the development of a technology that supports: (1) the use of higher strength reinforcing; (2) the stability of thinner concrete sections; and (3) a means of limiting strains in the compression region of panels." He referred to future PRESSS research results (which were not identified by the search done for this literature survey).

Type of Study: From Nakaki and Englekirk [1991]: organizational meeting for experimental and analytical work.

Abstract: "In April 1991, a series of industry seismic workshops were conducted by the Precast/Prestressed Concrete Institute (PCI). The primary objective of these workshops was to seek industry input into the Concept Development and Connection Classification Projects of PRESSS (Precast Seismic Structural Systems) Phase 1. The participants in these workshops consisted of precast concrete producers, design engineers, and contractors. Several conceptual designs were presented by the PRESSS researchers and critiqued by the workshop participants. This paper describes the results of the workshops as well as the review by the PRESSS Applications Advisory Committee, which recommends concepts worthy of future development by the PRESSS research teams."

PRESSS System Concepts: "The PRESSS research team decided early to develop system concepts and connections ideas that would apply to real buildings, rather than being just theoretical ideas. In this way, the economics of a system concept can be compared to other building systems during the initial phases of the research. In order to achieve this, two building functions were chosen for the study: (1) A four-story office building [see paper for typical floor plan]; and (2) a six-story hotel [see paper for typical floor plan]..."

"Three different frame system concepts were developed by the PRESSS team for the office building and presented to workshop participants." These concepts included: (1) post-tensioned frames, (2) cladding system; and (3) distributed frame systems. For the cladding system, "All seismic loads are resisted solely by the perimeter cladding system, and consequently may be used with any interior gravity load carrying system. Both loadbearing and non-loadbearing panels were considered." In addition to frame systems, wall systems were considered, including post-tensioned bearing/shear wall systems and reinforcing bar bearing/shear wall systems.

"Of primary consideration in developing the systems... was the interaction between connection type and system performance. As the researchers initially developed the systems, there was extensive interaction between the concept development and connection research teams to develop systems and connections that were compatible and would achieve the intended system behavior..."

"...Connector ductility must be considered as a variable in the design process which then influences the available system ductility and required seismic design loads. This is conceptually the same at the current *UBC* seismic design practice for both cast-in-place concrete and steel in the use of R_W factors and the prescriptive detailing requirements for each of the systems... In order to keep precast concrete as flexible a construction material as it has been, the PRESSS design recommendations should include methods for verifying acceptable connector and system behavior for different design load levels, in addition to any prescriptive detailing requirements that may be provided for systems with specified R_W factors."

Results of Workshop: For window wall cladding panels, "The panels will be non-loadbearing, and different shear connectors between panels will be investigated as the yielding element that limits the forces in the structure. All other connections will be standard architectural precast concrete connections,·i.e., bearing, lateral and in/out connections). The choice of non-loadbearing panels was made to maximize the feasibility of this system with various panel configurations that may be chosen by the architect." The figure in the paper shows an elevation of a 4-bay by 4-story frame. Each panel extends from column-to-column and from floor-to-floor. No connection details are given.

Future Research: For cladding systems, "A seismic load resisting system consisting of only the perimeter non-loadbearing cladding system will be developed. This can be used with any interior gravity load carrying system. The panels will be detailed to behave elastically, with all of the energy dissipation provided by panel-to-panel connectors." No details of the panel-to-panel connectors were given in the paper.

4.7a RESEARCH GROUP: School of Civil Engrg, Georgia Institute of Technology, Atlanta, Georgia

Topic of References: Analytical and experimental studies of ambient response of buildings clad with precast concrete cladding panels.

References: • Palsson and Goodno [1982]. "A Degrading Stiffness Model for Precast Concrete Cladding." • Goodno, Palsson, and Pless [1984]. "Localized Cladding Response and Implications for Seismic Design." • Palsson, Goodno, Craig, and Will [1984]. "Cladding Influence on Dynamic Response of Tall Buildings." Goodno and Palsson [1986]. "Analytical Studies of Building Cladding." • Palsson and Goodno [1988]. "Influence of Interstory Drift on Cladding Panels and Connections." • Craig, Goodno, Keister, and Fennell [1986]. "Hysteretic Behavior of Precast Cladding Connections." • Goodno, Meyyappa, and Nagarajaiah [1988]. "A Refined Model for Precast Concrete Cladding and Connections." • Craig, Leistikow, and Fennell [1988]. "Experimental Studies of the Performance of Precast Concrete Cladding Connections."

From Palsson and Goodno [1982]:

Abstract: "Presently, the lateral stiffness of heavy precast cladding and other nonstructural elements is usually ignored during design. However, earlier studies have shown that cladding systems can make a considerable contribution to the total stiffness of a structure, and furthermore that it may not always be conservative to ignore this contribution... Here, earlier studies are continued and a hysteretic model is introduced which governs the assumed behavior of heavyweight cladding for lateral interstory motions. The dynamic response of a [25 story steel-framed] highrise office tower is compared for three cases; (1) no cladding stiffness contribution; (2) full cladding stiffness contribution; and (3) degrading stiffness of cladding assuming hysteretic behavior of panels and connections."

Conclusions: "The results presented... demonstrate the potential influence of cladding stiffness on structure response to moderate ground motion. Due to increasing construction costs and the widespread use of heavyweight precast concrete facades for modern buildings, the potential stiffness contribution from the curtain wall and other nonstructural elements needs further study. In particular, laboratory tests should be performed to define the actual cyclic behavior of a full-scale cladding panels with a variety of connection details. The stiffness model presented here is felt to represent a first approximation to the cyclic hysteretic properties of heavyweight cladding. However, measured test data together with improved analytical models for both structure and cladding need to be developed for follow-on studies.

"Response comparison have been made for on ground motion loading only. Earthquake loadings possessing a variety of different durations and spectral characteristics must also be considered in subsequent investigations.

"Ultimately, investigations of cladding performance are expected to lead to increased knowledge of panel and connection forces for various levels of interstory motion. On this basis, improvement in design of cladding for earthquake loadings will result, leading to greater safety and economy in modern building construction."

From Goodno, Palsson, and Pless [1984]:

Abstract: "Overall and localized response models for cladding are presented in a study of the potential influence of heavy precast concrete cladding on the dynamic properties and seismic response of a medium highrise office building. Cladding was found to alter interstory drift and framing member forces in the supporting framework. Current recommendations for isolation of cladding through use of slotted connections may not perform as intended."

Conclusions: "The... studies were performed to determine the effects of cladding on overall structural lateral stiffness, on interstory drift due to earthquake ground motion, and on connection force levels in a frame-panel system subjected to interstory displacements. Results of the studies of overall structure response using the slotted connection model demonstrated that cladding stiffness can alter peak interstory drift values substantially for selected ground motion cases. For the localized response model investigation, the... results showed that the modified rather than the PCI recommended support conditions were best suited for use when connection forces must be kept at low levels for interstory racking of brittle facade components. Further studies of heavy cladding, including laboratory test of panels and connections, will lead to refinements in the cladding models and to improved understanding of force transfer to cladding due to interstory floor motions. Ultimately, studies of cladding behavior are expected to result in better procedures for design of panels and connections and improved performance of cladding in earthquakes."

From Palsson, Goodno, Craig, and Will [1984]:

Abstract: "Precast concrete panels form attractive facades for steel frame buildings and are generally regarded as non-structural by structural engineers. However, panels have been found to add lateral stiffness until their capacity or that of their connections is exceeded. Consequently, the computed dynamic response based on a model of the structural faming alone may be quite different from that experienced by the actual structure.

"As a case study, the influence of precast concrete panels on lateral and torsional stiffness of a 25-story building was investigated. The effect of cladding on dynamic properties and linear seismic response was explored by varying panel stiffness. Cladding stiffness was added to the bare frame model until analytical frequency values matched vibration test results. Then, using the cladding stiffness values obtained, an accidental eccentricity between the centers of mass and rigidity at each floor level was imposed and linear seismic response computed. Torsional response effects were increased substantially. Finally, a modified cladding panel connection was developed based on previously-reported studied for panelized construction. The influence of the proposed connection on overall structural response was determined for different ground motion inputs."

Conclusions: "The studies... have confirmed earlier reports in the literature which suggest that the exterior facade is a participating structural element, in spite of design assumptions to the contrary. Building frequencies and dynamic response predictions were found to be appreciably affected by cladding panel lateral stiffness and damping effects for the prototype structure. In addition, a comparison of results for the extremes of fully clad and unclad models demonstrated that it may not always be conservative to neglect the additional stiffening contribution of heavyweight cladding-connection systems. Neglecting cladding effects may be unconservative because dynamic characteristics of the overall structural model can be altered to such a degree by added stiffness that the sensitivity of the overall structure to certain earthquake loadings may be increased substantially (at least until the cladding has been sufficiently damaged to render it ineffective).

"A cladding connection possessing elasto-plastic behavior and stable hysteretic response was presented and was observed to be effective in reducing overall structural response for the two ground motion cases considered. However, the question of whether the potential advantages of the brake pad type of connection outweigh its disadvantages has bot yet been fully addressed in these studies. Future investigations must consider relative costs, complications in design and construction, and performance under other loading conditions. Most importantly, a fundamental program of laboratory testing of heavyweight cladding and connection systems in common use today must be initiated so that much-needed data can be obtained to guide future analytical and experimental studies of cladding performance in modern highrise building construction.

"In future studies, the influence of cladding on design of the overall structural frame for lateral forces must be more fully explored. Ultimately, more rational design procedures for cladding and other non-structural elements are expected to result from these investigations."

From Goodno and Palsson [1986] (Note: Palsson and Goodno [1988] is similar):

Abstract: "Four different analytical models are presented for architectural precast cladding on highrise buildings. An interstory shear stiffness model, an incremental failure model, a hysteresis model and a slotted connection model were used to study cladding performance from the linear elastic range up to failure. The models were added to a finite element model of the structural frame of a 25-story steel frame office building to account for the lateral stiffening effect of the building facade. Cladding stiffness values were based upon a comparison of measured and computed vibration frequencies for the building. Results of the study include peak interstory drift plots and a representation of the cladding failure state for the case of moderate earthquake ground motion input. Cladding stiffness was found to have a substantial effect on building dynamic properties and linear seismic response."

Conclusions: "Four different analytical models were presented to account for the influence of architectural precast concrete cladding on seismic response of a medium highrise office building to moderate ground motion. The interstory shear stiffness model was simple in concept and application. Vibration frequencies and peak time-history response values for the overall building model were altered substantially by addition of cladding stiffness with this model. Next progressive failure of cladding was represented by an incremental failure model. Failure was defined as graduated loss of cladding stiffness as allowable drift limites were exceeded at different levels on different structure faces during the applied ground moton. Torsional response of the overall building increased greatly as the capacity of cladding elements was exceeded at different points on the facade. Following the failure model, a hysteresis model for cladding was presented but its properties were established without benefit of experimental data. The model included elastoplastic and shear-slip behavior but load degradation was not considered. Drift envelopes for the hysteresis and interstory shear stiffness models were nearly the same. Finally, a slotted connection model was presented based on current recommendations for attachment of precast panels. As with previous model, building response was substantially different from the bare frame model when the slotted connection model was used to predict peak interstory drifts..."

From Craig, Goodno, Keister, and Fennell [1986]:

Abstract: "A preliminary experimental study is presented which investigates the behavior of connections for architectural precast concrete cladding panels. The principal objective is to quantify the lateral stiffness capacity, energy dissipation properties, and ductility of representative precast cladding designs used on several highrise buildings. Hysteretic models will be developed on the basis of test response in the linear and nonlinear stages of behavior. Test results are presented for three $^3/4$ in. diameter wedge-type inserts subjected to direct pullout. The reaction frame is designed to perform moment, shear and combined shear/pullout tests. This study is expected to lead to a better understanding of precast connection behavior and performance and thereby aid in developing a more rational connection design procedure."

Conclusions: "The inserts experienced highly linear strain and displacement results up to 10,000 lbs. In this load range, the hysteresis loops were narrow, indicating little energy dissipation. In the load cycles from 10,000 lbs. to failure at 11,000 to 12,000 lbs., the samples experience nonlinear behavior with changing slopes of load versus displacement and load versus strain curves. Also, in these final cycles, the hysteresis loops became very large indicating greater energy dissipation possibly due to reinforcing steel deformation.

"The method of failure of the inserts was unsatisfactory for practical situations because of its suddenness and catastrophic fracturing of concrete. It was noticed after failure that the reinforcing steel underwent considerable deformation. This deformation was probably the reason the strain and deformation plots underwent pronounced changes at 10,000 lbs. with strain and displacement plots changing to wide hysteresis loops. The reinforcing steel used with the inserts was only 10 in. long and lengthening it would most likely add considerably to the ductility and survivability at loads above 10,000 lbs. To maximize survivability, it would probably be best to tie the insert into panel flexural steel instead of adding bars just for the insert alone. This would increase the insert steel's embedment providing better continuity of the insert to the panel. This continuity would induce a connection failure by a ductile structural failure of the panel instead of a sudden brittle localized fracture noticed in these tests. Specific recommendations for modifications to the insert support detail await further testing involving other loads, load combinations and panel support arrangements."

From Goodno, Meyyappa, and Nagarajaiah [1988]:

Abstract: "A wide variety of panel and connection designs have been used for architectural cladding on modern buildings, often without consideration of the possible interaction of the structure and its facade during lateral interstory motions due to wind and earthquake. However, past studies have shown that heavyweight cladding can measurably influence the lateral stiffness, dynamic properties and response of highrise buildings. The objective of the study... was to develop a more refined model for a portion of a typical heavy cladding system and to use it in a seismic response evaluation of a case study [25-story steel-framed] building to determine cladding panel, panel insert, and connection forces due to interstory drift motions. The overall cladding system was envisioned as comprised of a series of superelement models which detailed the localized response of representative portions of the facade. The superelement model included the effects of separate connection and insert models developed in conjunction with laboratory experiments on wedge and loop ferrule inserts embedded in concrete slabs. Individual components of the model and sample response values are described [in the paper]."

Conclusions: "The primary objective of these studies has been to obtain a better understanding of the behavior of one of the major nonstructural systems present in modern buildings, the exterior facade. The financial loss associated with nonstructural damage in modern buildings due to earthquakes often exceeds the cost of repair of structural damage and this provides the primary motivation for these studies. Heavy cladding systems offer the potential for increased stiffness and damping in buildings, but their interaction with the structural framing under seismic motions must be better understood if they are to be designed properly. Improved analytical modelling, combined with comprehensive experimental testing programs and increased efforts to document nonstructural damage more thoroughly in actual earthquakes, are expected to provide insight into cladding behavior to guide future design improvements."

From Craig, Leistikow, and Fennell [1988]:

Abstract: "An experimental program involving the design and execution of laboratory tests of cladding panel connection subsystems is described. The program was designed and carried out as part of larger study that included both analytical and experimental modelling of cladding system. The design of the test program is described and the results of studies of bolt-insert and ductile rod push-pull connections are presented. Linear and nonlinear constitutive characteristics are described for these connections. The use of this information in detailed analytical models of cladding on typical highrise buildings [part of the larger study] is outlined. The results indicate that commonly-used connection design may be susceptible to low-cycle fatigue failure. This observation suggests that current connection design practice may need to be reevaluated for adequate performance over several earthquakes."

Conclusions: "A program of experimental testing and analytical modelling is currently underway to provide quantitative information about the performance of a representative set of connection designs. While the initial focus of this work is on the connections, it is anticipated that continuing work will address the properties of the cladding panels, both by themselves and in association with the connections. Specific conclusions at this point in the program can be stated as follows;

1. "Experimental tests, along with the analytical models have confirmed the basic behavior assumed for push-pull ductile rod connection designs. Measured stresses and deflections agreed well with linear elastic beam models. These models were able to accurately predict the onset of

inelastic behavior at large levels of [in-plane] displacement. The most significant result of these tests was the observation of low-cycle fatigue of the ductile rod. In all 8 cases tested, the rods experienced low-cycle fatigue cracking at one or both ends for displacement amplitudes up to but not exceeding typical (*UBC*) code provisions for interstory drift. In one half of the cases complete fracture occurred at one of the other end within 25 displacement cycles. This is a cumulative effect and could be reached after several strong earthquakes over a period of time.

2. "Initial multi-axis tests of several rigid connection types [have] yielded information for simple linear and nonlinear models. Agreement with [a] detailed linear finite element model incorporating nonlinear gap elements was generally poor but stated design capacities were found to be conservative. Further tests will be needed to complete more accurate models.
3. "The authors are confident that rational engineering principles can be applied to the design of cladding systems on buildings. It is possible that heavy cladding systems will be used in the future for both lateral stiffening and increased damping in builds. However, it must also be recognized that improper or inadequate design of building cladding may lead to failures that could have a detrimental effect on the overall performance of structures during earthquakes."

4.7b RESEARCH GROUP: School of Civil Engrg, Georgia Institute of Technology, Atlanta, Georgia

Topic of References: Analytical and experimental studies from post-earthquake observations of the 1985 Mexico City earthquake.

References: • Goodno, Craig, and Zeevaert-Wolff [1989a]. "Behavior of Architectural Nonstructural Components in the Mexico Earthquake: Final Progress Report." • Goodno, Craig, and Zeevaert-Wolff [1989b], "Behavior of Heavy Cladding Components." • El-Gazairly and Goodno [1989]. "Dynamic Analysis of a Highrise Building Damaged in the Mexico Earthquake including Cladding-Structure Interaction." • Pinelli and Craig [1989]. "Experimental Studies on the Performance of Mexican Precast Cladding Connections." • Pinelli, Craig, and Goodno [1990]. "Development and Experimental Calibration of Selected Dynamic Models for Precast Concrete Cladding Connections."

From Goodno, Craig, and Zeevaert-Wolff [1989a]:

Type of Study: Experimental and Analytical.

Project Scope: "This research program is aimed at acquiring much-needed data on performance of nonstructural building elements in a recent major earthquake. This primary objective of this study is to develop a functional understanding of the role played by nonstructural cladding elements in the structural performance of buildings under severe ground motion conditions. Specifically, the concern is with both the actual and potential contributions of cladding to the lateral stiffness under normal loading conditions and potential energy dissipation (damping) that can be developed under severe loading conditions. Secondary objectives include an assessment of the appropriateness of existing code provisions related to cladding and the identification of potential modifications or extensions that could lead to improved performance.

"The research program consists of a combined field study of building cladding performance during the 1985 Mexico earthquake and supporting analytical and experimental studies of cladding connection systems typical of practice in Mexico City. This research effort is broken down into three phases: Phase I - Nonstructural damage survey and evaluation for selected buildings in Mexico City; Phase II - Laboratory testing of cladding connections representative of Mexican practice; and Phase III - Analytical evaluation of case study buildings for cladding-structure interaction effects.

"Laboratory testing and analytical studies of cladding connection designs typical of U.S. practice have been under way by the authors for several years. The present study of the behavior of architectural cladding systems in the Mexico earthquake is complementary to this work. The data gathering, laboratory testing, and analytical phases outlined above are designed to provide a balanced and coordinated attack on the problem of nonstructural performance in earthquakes and to extraction of as much useful information as possible for the benefit of both Mexico and the United States."

Summary of Findings: Phase I - "...A paper presenting the results of the Phase I efforts has been prepared Goodno, *et al.* [1989b] and published in EERI's *Earthquake Spectra.*

The study involved the initial identification of 30 or more buildings that were either known or expected to have experienced noticeable damage to cladding systems. A combination of visual observation, personal contact and review of available damage survey was used to establish an initial list of 25 candidate buildings meeting the following criteria: (1) buildings with relatively extensive precast or GFRC cladding systems; (b) structures in the 10-20 story range; and (c) buildings for which both structural design and as-built information was available, or for which on-site inspections could reveal the latter information.

"The last requirement was the most important and eventually eliminated several potentially interesting buildings. The 25 buildings were reviewed and reduced to 12 structures that meet all of the above conditions. Nearly complete architectural and structural drawings were obtained for 4 of these buildings, and one was finally selected for detailed study of both the structure and the cladding as part of the Phase II and III work. In addition to the damage surveys results of the Phase I study also include a brief review of cladding design practice and representative examples of connection designs in the Mexico City region."

The Phase II laboratory tests and Phase III analytical studies are described briefly in this paper. More detail is provided in the next two references given in this section. The Phase III results are presented first, because the paper cites references from the Phase II paper.

From El-Gazairly and Goodno [1989]:

Type of Study: Analytical.

Abstract: "Investigation of the effect of cladding on the seismic response of an existing structure is presented. The selected building is a twelve story reinforced concrete frame structure severely damaged during the 1985 Mexico earthquake. The street face of the structure was clad with heavy precast concrete spandrel panels. Additional cladding was used to enclose the columns at the front corners of the building. A three-dimensional tier building model was developed to study the effect of cladding on the fundamental frequencies, mode shapes and seismic response. The lateral stiffness of the cladding, along with an appropriate representation of the structure foundation to account for soil-structure interaction, were include in the overall model. The cladding panels were modelled using finite elements and panel connections were represented with springs having variable properties. Linear response spectrum analysis was performed using the SCT record from the Mexico earthquake. Force levels developed in the cladding connections were examined."

Observed damage: "The reinforced concrete structure experience severe cracking at waffle slab-column connections including several cases of punching failure. The water tank on the roof failed resulting in a partial collapse of the rear part of the structure. Masonry infill walls suffered diagonal shear cracks and lost contact with floor slabs. The precast column cover panels were cracked at the location of the plate inserts in the panels; the cracks were visible on the front face of the building at almost every level but no panels were ejected from the structure. The spandrel facade panels along the front face of the building were not damaged because they were not connected between successive floors. The building was undergoing extensive repair and rehabilitation when the damage survey was conducted."

Analytical Studies: The building used in the case study has "a twelve story reinforced frame with shear walls and heavy precast cladding. The first two floor levels are used for parking, while the rest of the floor are reserved for commercial offices." Floors 2 and 3 are 5 by 6 bays, and the upper floors are 3 by 6 bays in plan. In the longer building direction, the bays are typically 7.33 m. and are located over the northern-most lower story bays; in the shorter building direction, the bays are 9.00 m. The upper story plans are asymmetrical, with the vertical circulation shafts located outside the 3 by 6 bays, but built integrally with the longer western perimeter of the building.

"The [building] has 20 cm (8.16 in.) thick masonry walls along its two sides and along the back. However, 10 cm (4.08 in.) thick masonry walls are used as interior partitions in story levels 3 to 7. The front [east facade] of the building is clad with heavy precast concrete spandrel panels, while additional cladding is provided around the columns at the front corners of the building."

Further information on the structural framing, member sizes, etc., can be found in the paper.

"...The cladding design consists of column covers at each front corner of the building. Column covers are assembled from individual panels, while spandrel panels are comprised of individual sloped elements... Each type of cladding component is support on the floor slab. The connection anchors are fabricated from steel plate with welded reinforcing bars arranged to conform to panel geometry. The weld plates are attached directly to embedded inserts in the slabs using simple clip angles, rectangular bar stock, or direct welding."

Description of Analytical Models: The interested reader is referred to the paper for a detailed description of the analytical model, including foundation representation, stiffness of structural elements, cladding model, and mass model.

Software: GTSTRUDL

Analysis Types: Analysis included eigenvalue and response spectrum.

Ground Spectra: "The nearest recording station to the building site was the SCT (Secretaria de Comunicaciones y Transportes) station located at latitude 19.393 North and longitude 99.147 West in the old lake bed area... Elastic response spectrum analyses of the structure were performed using the SCT record with 5% damping in N90W and S00E directions."

Response Quantities, Selected Results, and Implications for Future Analysis: From response spectrum analysis: peak interstory drifts and maximum floor displacements. "The computed peak interstory drifts were used to determine the force levels in the cladding connections. The procedure was to define these drifts as specified displacements in the column cladding finite element model developed earlier using GTSTRUDL. Forces in cladding connections were calculated for each mode, then the model contributions were combined using SRSS and CQC to obtain the peak values of these forces. Standard cladding connections can usually sustain applied forces in the range of 44.4 kN (10 kips). In this preliminary investigation, the magnitude of the maximum forces developed in the cladding connections were found to exceed the connection capacities by up to a factor of ten. While this level of response would certainly explain the observed failure of the connections, it also violated the original assumption of linearity in the model and analysis. Hence, a time history analysis, in which the time and location of connection failures is detected and appropriate modifications are made to the overall model, is necessary and will comprise the next stage of the investigation. Partial cladding failure can then be included in the model and its effect on subsequent response accounted for in the analysis. Here, the computed stiffness properties of the cladding connections will be altered on the basis of their measured nonlinear hysteretic behavior. Additional effects, such as the failure of masonry infill walls and failure of selected slab-column joints, must also be accounted for in future refinements in the overall model."

Summary and Conclusions: "...Linear dynamic response of the structure was computed... Different stiffness values for the cladding connections were used to study the effect of heavyweight cladding on the response of the building. The tabulated results showed the importance of cladding in altering the dynamic properties and behavior of a case study building damaged in an actual earthquake. It also confirmed earlier reports which suggest that the exterior facade is a participating structural element, in spite of the design assumptions to the contrary.

"Computed building frequencies and dynamic response were found to be appreciably affected by cladding stiffness effects for the prototype structure. An increase of 30% to 49% in the lower frequencies was observed for different values of connection stiffness. Peak interstory drifts and maximum roof displacements were greatly affected as a results of the cladding contribution. Forces at column cladding locations were determined to largely exceed the cladding connections capacity. This confirms the observed failure in these connections during the earthquake. No damage was reported in the spandrel cladding panels at the front face of the building because of their discontinuity between successive floor. Soil-structure interaction has a significant effect on structure response, as expected."

From Pinelli and Craig [1989]:

Type of Study: Experimental.

Abstract: "Many buildings in Mexico City suffered extensive damage to cladding during the 1985 Mexico City earthquake. In the present study, laboratory tests were carried out to evaluate the performance of precast cladding connections typical of Mexican practice. A building which suffered cladding damage was selected, and its cladding connections were identified as being fabricated around weld-plate type inserts with some variations. Connection inserts typical of this building were tested in the laboratory to determine shear and moment capacities, linear and nonlinear characteristics of the hysteretic behavior, energy dissipation properties, and ductility. The results of the tests are presented and discussed. The direct use of these results will be in continuing analytical modelling of this and other buildings to determine cladding performance, both actual and potential, during strong ground motion. Ultimately, this study should lead to recommendations for better cladding design procedures in both Mexico and the U.S."

Experimental Program:

Description of Test Specimens: "The specimens used for the inset tests were of two basic types, one 36 in. x 28 in. x 8 in. [920 mm x 710 mm x 200 mm] and the other a smaller 28 in. x 20 in. x 8 in. [710 mm x 510 mm x 200 mm] configuration, both reinforced with wire mesh. The test specimens were purposely designed to be as thick as possible and were anchored firmly enough to preclude significant bending action in order to focus entirely on insert behavior. ...the specimens were designed to accommodate several types of embedded steel inserts placed on the perimeter edges. The five different configurations employed for the present tests (are shown in the paper). The specimens were fabricated at a local precast manufacturer using nominal 4000 psi concrete." Figure 4.1 (taken from Fig. 1 in the paper) shows the configurations.

Type of Loading: Quasi-static for single axis in-place shear load (parallel to specimen which simulates gravity of in-plane lateral or racking loads), single axis bending about an axis parallel to specimen which simulates out-of-plane bending loads, and a combination of these. "Since the principal concern is with connection performance under earthquake loading, the measurements were carried out under cyclic loading conditions. A loading cycle is defined here to consist of incremental increases in load in one direction to a maximum, return to zero in equal unloading increments, followed by the same procedure in the reverse load direction. The loading in either direction is referred to as a positive or negative half-cycle. In each case, the inserts were subjected to several cycles of loading of increasing magnitude. Cycles of pure shear loading were alternated with cycles of pure moment loading. Shear cycles were applied in 250 lb (1.1 kN) and 500 lb (2.2 kN) increments. Moment cycles were applied in 500 in-lb (56.5 N-m) and 1000 in–lb (103 N-m) increments. For each cycle, either the LVDTs or potentiometers were used to record the deformation across the insert (translation, rotation, twist) at the insert-connection interface." Figure 4.2 (taken from fig. 2 in the paper) shows the loading schematic.

Main Findings: In Figures 4.3a, 4.3b, 4.4a, 4.4b, and 4.4c (taken from figs. 7a, 7b, 9a, 9b, and 9c, respectively, in the paper) graphs are given of translation versus deformation and of moment versus rotation for selected connections.

Conclusions and Recommendations: "Seven weld plate inserts, all typical of Mexican practice, but of varying geometry and configuration were tested. The influence of the location of the inserts in the cladding panel was also investigated. The inserts were tested under cyclic shear and cyclic moment loads. In both types of tests, the inserts exhibited a pronounced hysteretic behavior characterized by pinching and bilinearity. This behavior can be explained by the interaction between the steel insert and the surrounding concrete. At low levels of load, the stiffness is provided primarily by the concrete surrounding the insert. As the magnitude of the load increases, the insert is blocked against the concrete and further stiffness is provided by the steel propperties of the insert. As the magnitude of the load cycles was increased still further, the concrete began to deteriorate. In many cases the concrete failure was sudden and brittle, especially in the case of inserts located at the corner of a specimen where the concrete suffered from a lack of confinement. Once the concrete failed, the connections began acting like a hinge with the steel insert experiencing large displacements into the cracked concrete environment and the only stiffness being provided by the steel insert. Ultimately, the connection failed by a total collapse of the concrete or a failure of the weld between the steel plate and the rebars.

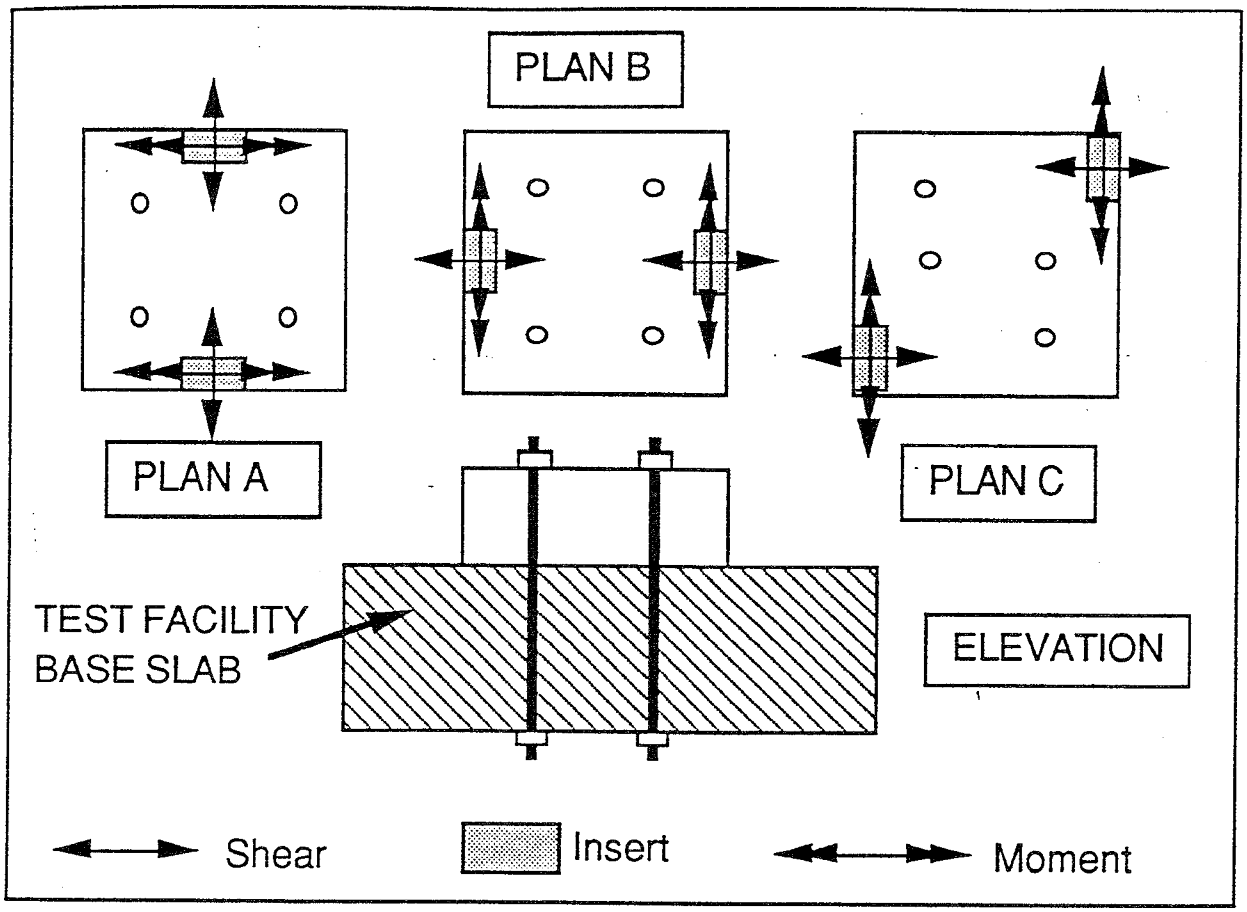
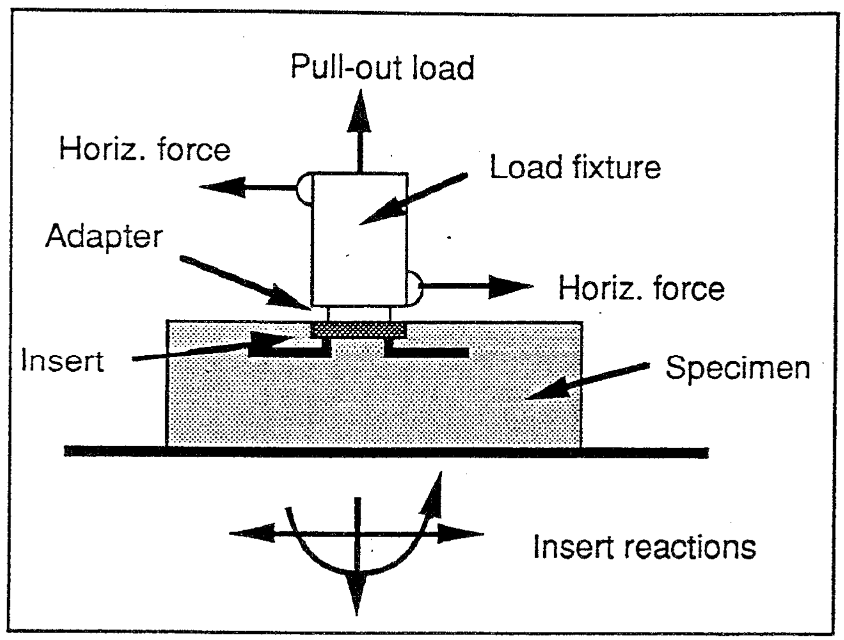


Figure 4.1. Test specimen design showing corner and edge configurations (from Pinelli and Craig [1989]).

Figure 4.2. Loading schematic (from Pinelli and Craig [1989]).



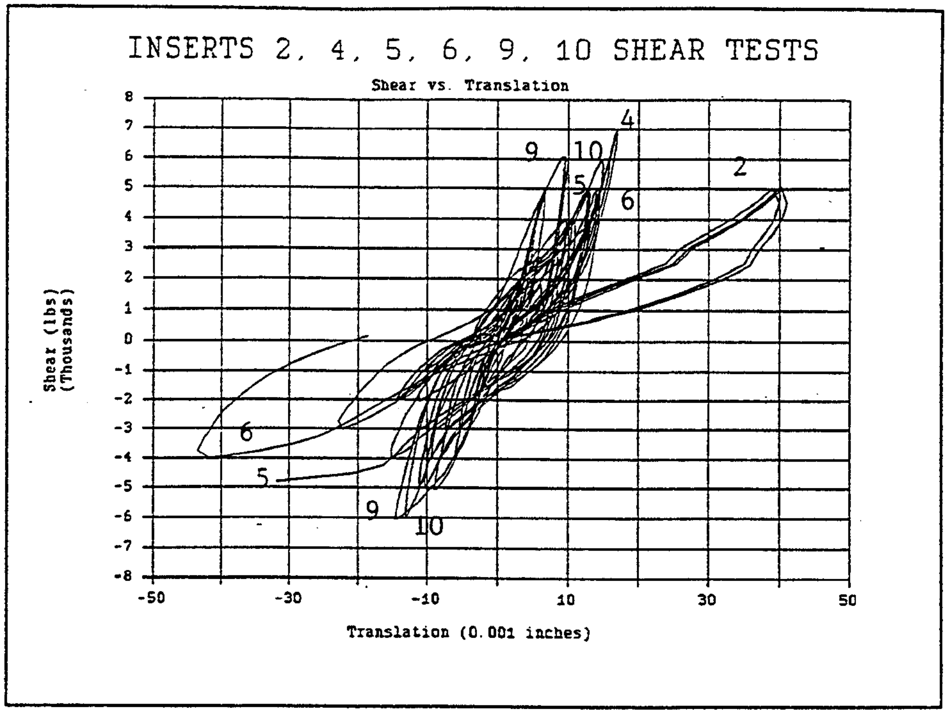


Figure 4.3a. Shear deformation for all inserts (from Pinelli and Craig [1989]).

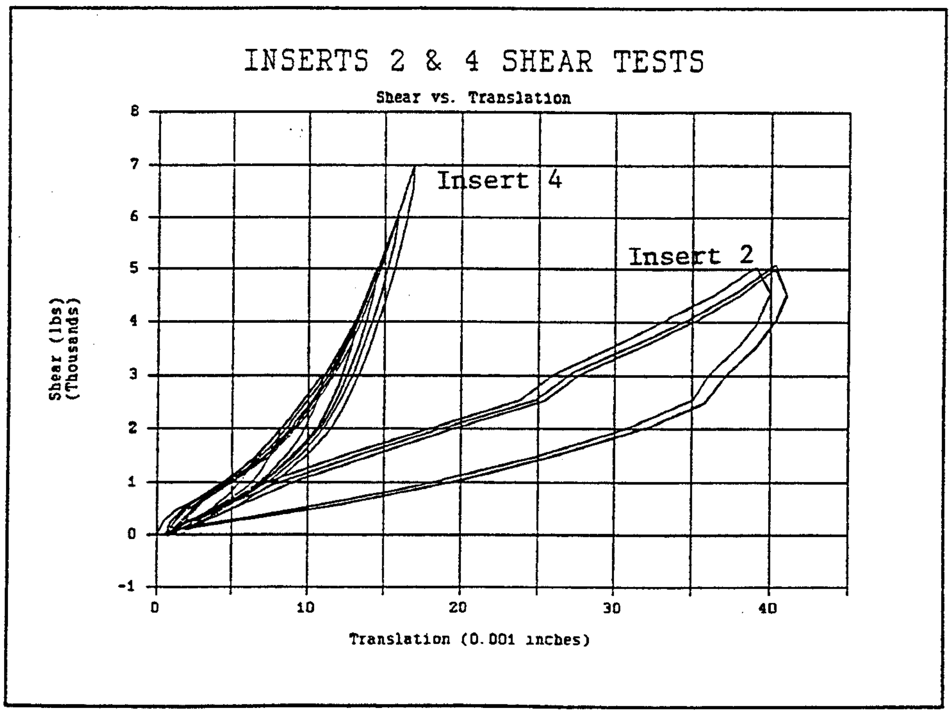


Figure 4.3b. Shear deformation for inserts 2 and 4 only (from Pinelli and Craig [1989]).

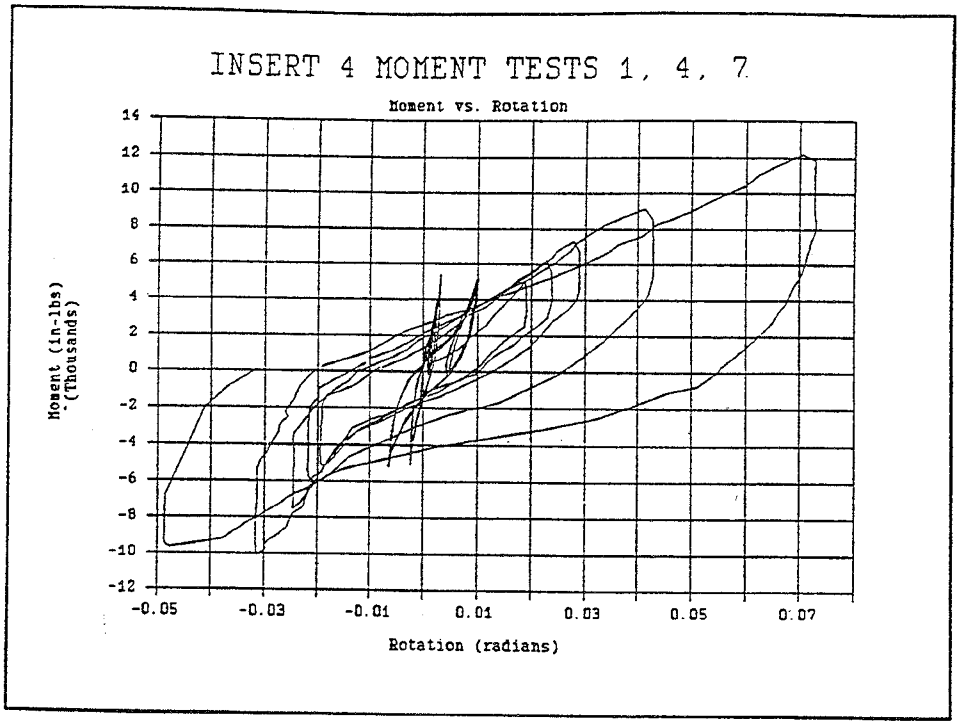


Figure 4.4a. Moment deformation for insert 4 (from Pinelli and Craig [1989]).

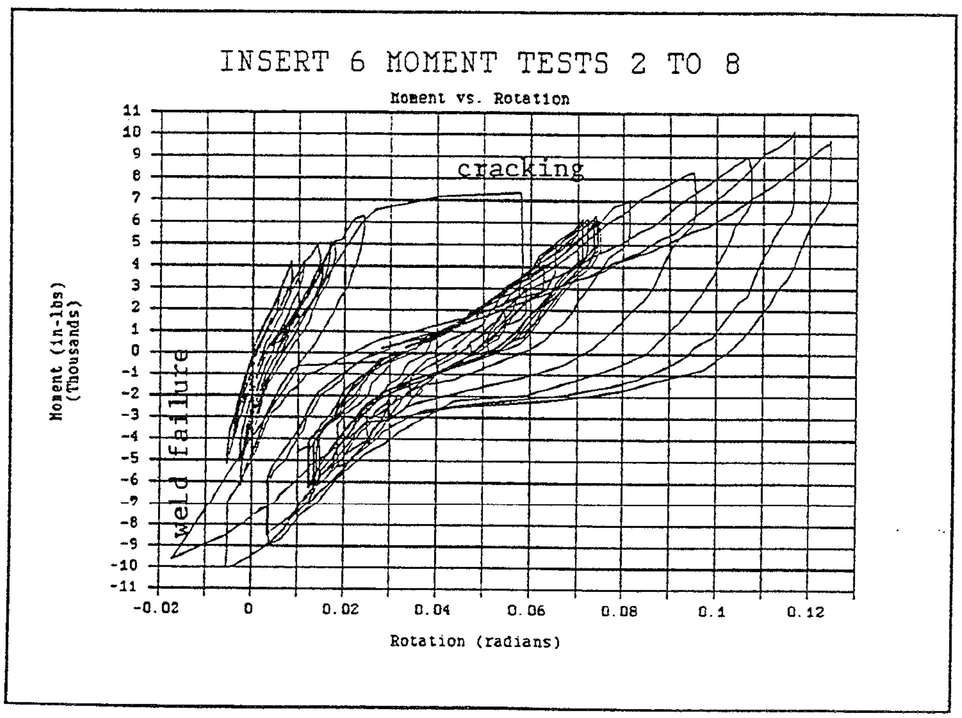


Figure 4.4b. Moment deformation for insert 6 (from Pinelli and Craig [1989]).

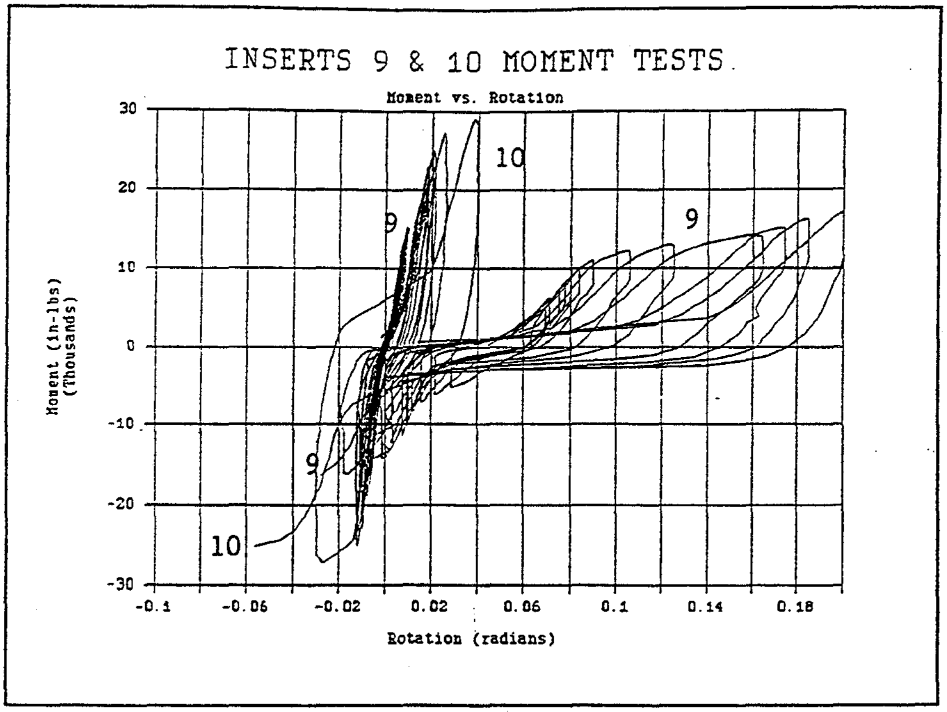


Figure 4.4c. Moment deformation for inserts 9 and 10 (from Pinelli and Craig [1989]).

"As expected, inserts located along the edge of the specimens behaved better than the inserts located at the corners, in the sense of developing higher strength values and higher stiffness. However, it was shown that the corner insert with two rebars in each direction also exhibited an acceptable behavior, especially in the ability to restrain deterioration of the concrete.

"The values of ductility were found to vary substantially with the type of insert, its location in the concrete specimen, the load type, and the magnitude of the load cycles. But in all cases. a tendency toward an increase in ductility with increase in the magnitude of the load was observed. More specifically and as expected, the ductility increased dramatically once the concrete started to crack.

"A more detailed evaluation of the energy dissipation characteristics of the test results has yet to be completed. However, a simple examination of the shape of the hysteresis loops shows the decisive influence of the concrete deterioration on this aspect of the mechanic behavior of the connection. If one considers the amount of damping provided by the connection to be related to the area under the hysteresis loops, it can be observed that once the concrete failed, this area increased dramatically. Such damping characteristics would certainly be considered a positive aspect of the connection if they were not obtained at the expense of the cracking and crushing of the concrete. Future design should try to enhance this behavior, but they should also minimize the deterioration of the concrete and maintain the connection integrity.

"Additional testing of similar inserts and inserts of different types is planned for the near future. Analysis of the results of the present tests, of earlier tests, on other types of inserts, and of tests yet to be carried out will lead to the definition of an analytical nonlinear hysteretic model for the cladding connections. Such a model will allow the refinement and calibration of overall building models. Such an improved representation will permit more accurate investigations of the effect of cladding on energy dissipation, and torsional and lateral response of buildings.

"Ultimately, these studies should result in improved design methods for cladding connections. The objective is to formulate design guidelines that will result in a new generation of cladding connections that will incorporate the necessary properties of strength, stiffness, ductility, and damping to allow the cladding to be an integral and accepted part of the lateral load resisting structural system of a building. The end result will then be safer and more economical structures."

From Pinelli, Craig, and Goodno [1990]:

Type of Study: Experimental.

Abstract: "Research in recent years has pointed to the potential role that heavyweight cladding can play, when properly designed, in providing ductility and energy dissipation to the overall building structure during strong ground motions. Extensive analytical modelling carried out by the authors and others have pointed to the critical role that cladding connections can play in this process. To more accurately evaluate the potential of this design strategy, laboratory tests were carried out to evaluate the performance of typical precast cladding connections. Cladding connections fabricated around weld-plate type inserts with some variations were tested in the laboratory to determine shear and moment capacities, linear and nonlinear characteristics of the hysteretic behavior, energy dissipation properties, and ductility. The results of the tests are presented and discussed. The use of these results in the development of nonlinear constitutive models for cladding connections that can be used in continuing analytical modelling of buildings to determine potential cladding performance during strong ground motion is outlined."

Note: This paper appears to have much of the same content as Pinelli and Craig [1989].

4.7c RESEARCH GROUP: School of Civil Engrg, Georgia Institute of Technology, Atlanta, Georgia

Topic of References: Analytical and experimental studies of "advanced connections."

References: • El-Gazairly, Goodno, and Craig [1990]. "Analytical Investigations of Advanced Connections for Precast Concrete Cladding on Buildings." • Pinelli, Craig and Goodno [1992]. "Development of Advanced Connection Concepts for Precast Concrete Cladding." • Wolz, Hsu, and Goodno [1992]. "Nonlinear Interaction between Building Structural Systems and Nonstructural Cladding Components." • Goodno and Craig [1991]. "Modeling of Advanced Precast Concrete Cladding Connections for Seismic Design." • Goodno, Craig, and Hsu [1991]. "Experimental Studies and Analytical Evaluation of Ductile Cladding Connections." • Pinelli, Craig, and Goodno [1991]. "Hysteretic Connection Models for Load Resisting Precast Cladding Panels in Seismic Zones." • Craig, Goodno, Pinelli, and Moor [1992]. "Modeling and Evaluation of Ductile Cladding Connection Systems for Seismic Response Attenuation in Buildings." • Pinelli, Moor, Craig, and Goodno [1992]. "Experimental Testing of Ductile Cladding Connections for Building Facades." • Goodno, El-Gazairly, Hsu, and Craig [1992]. "Use of Advanced Cladding Systems for Passive Control of Building Response in Earthquakes." • Pinelli, Craig, Goodno, and Hsu [1993a, 1993b]. "Passive Control of Building Response Using Energy Dissipating Cladding Connections" and "Response to J.M. Cohen's 'Discussion of Passive Control of Building Response Using Energy Dissipating Cladding Connections.'" • Pinelli, J.P.; Craig, J.I.; and Goodno, B.J. [1994]. "Design Criterion for Energy Dissipating Cladding Connections."

Note: In this section, the abstract and conclusions only are included from each paper, unless otherwise noted. The interested reader is referred to the papers for further details.

From El-Gazairly, Goodno, and Craig [1990]:

Type of Study: Analytical.

Abstract: "The role that architectural precast cladding systems can and should play in the seismic response of building structures is addressed. Architectural precast is often dismissed as nonstructural and does not normally form part of either precast frame or wall-panel building construction. Past studies have shown that heavy cladding systems act to stiffen buildings during earthquakes and alter their predicted response based on a model of the bare frame alone. Cladding damage in the 1985 Mexico City and 1989 Loma Prieta earthquakes offered ample evidence of the active role of modern cladding systems in the lateral response of buildings. This suggests the possible use of precast panels and their attachments to the structure in an integrated building cladding system which provides both increased lateral stiffness and damping for the structures as a whole. Conceptual models for several advanced connection design that may offer improved energy dissipation, ductility and failure are presented. The concept must be investigated using an actual case study building to test its validity. Initial results of this on-going investigation are presented [in the paper]."

Summary and Conclusions: A dynamic analysis of a highrise building damaged during the 1985 Mexico earthquake including cladding-structure interaction effect has been presented.

The importance of cladding in altering its dynamic properties and behavior was demonstrated. However, the present model is valid only for the initial stages of the earthquake response when the building response is still linear. In on-going work, the measured nonlinear characteristics of the cladding connections are being incorporated into a nonlinear model of the entire structure. This investigation is expected to provide additional insight into observed structural and nonstructural failure patterns. Finally, this structure will be used to explore the potential benefits of advanced cladding connections which have improved stiffness, damping and ductility properties compared to present designs. Continued research is expected to results in better design methodologies for cladding and connection systems that improve the safety and performance of the cladding panels and their connections compared to present designs."

From Pinelli, Craig, and Goodno [1992]:

Type of Study: Experimental and Analytical.

Abstract: "Architectural considerations usually govern the selection of cladding facade panels. However, precast concrete cladding system if properly designed, could provide lateral stiffness, ductility, and energy dissipation to an overall building structure, especially during strong ground motions. Cladding connections play a critical role in this process, and the present paper explores how new concepts for cladding participation could be implemented through the development of advanced connections. An advanced connection is one which exhibits superior properties of ductility and damping resulting in high energy dissipation without failure in the event of a moderate or strong earthquake.

"The paper describes the different phases of the development of advanced connections in both their experimental and analytical aspects. Test fixtures, program and results are presented. Work to date has examined in detail the concrete insert component of a cladding connection and has resulted in the development of empirical and mechanical models. These models idealize the connection inserts as nonlinear rotational elements that incorporate the properties of stiffness degradation, pinching behavior, and related variations of the hysteresis loop area. The work is currently being extended to include the connection elements themselves, and as for the inserts the approach is to formulate both mechanical and empirical models on the basis of experimental measurements from laboratory tests. In all cases, agreement with the experimental results, model simplicity, and computational efficiency are the basis for the evaluation of the models which in turn will be used in continuing studies of full-scale building response."

Objective: "The goal of the research program is not to produce a single connection design but rather to develop a methodology that could be applied to different connection systems. This strategy is not unlike that which has evolved for eccentric bracing when employed to provide enhanced ductility in primary structural systems. Accordingly, it appears that the best course of action regarding the development of advanced connections is a gradual approach that consists of the following phases: (1) testing of conventional connection components; (2) analytical modelling of conventional connection components; (3) identification of possible sources of improvement in the areas of ductility, damping, and strength; (4) analytical formulation of the advanced connection model; (5) testing of advanced connection designs; and (6) calibration of analytical models for advanced connections."

Overview of Paper: The authors summarized the results from their research to date (see references listed above). The description of testing of conventional connection components includes the test fixture and specimens, test results, and damping measurements. The analytical modelling of conventional connection components includes an empirical approach and a mechanical approach.

Future Research: "Following the investigation of the cladding panel and RC frame inserts, a conventional connector design will be tested...

"Once hysteretic models have been defined for each of the three connection system components, the translational and rotational elements will be combined to simulate a complete connection system. Finally the three components (insert-connector-attachment) will be tested together and the analytical assemblage will be calibrated against the results of the tests. These tests and analyses should reveal the influence of each component on the behavior of the complete system in the nonlinear range. They will also identify the causes for potential enhanced or detrimental performance in the areas of damping, ductility, and strength..."

From Wolz, Hsu, and Goodno [1992]:

Type of Study: Analytical.

Abstract: "Results of past research suggest that a properly designed system of precast cladding panels and their connections could provide desirable increases in lateral stiffness and damping, thereby contributing to performance improvements and reduced cost for the structure as a whole. Extensive studies based largely on linear structural models have shown the potential benefits of this approach, but the results have also indicated that inelastic behavior of the connections can be expected. To better understand the nature of the nonlinear interaction between structural and nonstructural systems, as study of a simple building model was undertaken and preliminary results are presented...

"A six story space frame (1/4 scale), used in a recent analytical and experimental study at NCEER of an active control system for aseismic protection, was chosen for further investigation here. Initially, the frame was modeled using DRAIN-2D, then both the model and the program were modified to include a nonlinear cladding connection element. Both a bilinear and a degrading stiffness model were used to represent the hysteretic response of each connection element and to estimate the amount of energy dissipated during each time step. Parametric study of several different connection types is planned to aid in the selection of those to be tested in the laboratory in the experimental phase of the overall research program. Final results of the analytical investigation are expected to demonstrate the potential benefits of cladding as a passive control system which may be used, possibly in conjunction with other systems, to reduce overall structural response."

Assumptions and Analytical Modelling: "The connection element was developed specifically to represent the points of attachment between the panel and the structure. Typically, a cladding panel is secured by four attachments, one near each corner. The degree of influence a panel will exercise on the deformation of the structural member to which it is attached, depends on the fixity of these connections. For the study, the lower connections each consisted of three springs allowing for horizontal and vertical translation, as well as rotation in the plane of the panel. The horizontal (x-translation) movement of the flexible connections was tracked using either a bi-linear or a degrading stiffness model, while the two other displacements (y-translation, z-rotation) were assumed to remain in the elastic range. The distortion of the connection in the horizontal direction was calculated as the relative displacement between the nodes to which the connection is attached.

"Unlike the bilinear response model, the degrading stiffness response model accounted for the softening of the connection due to plastic deformation. The response model for the flexible connection was based on a force-displacement rather than moment-rotation relationship. The horizontal distortion of the connection was calculated as the relative displacement between the nodes to which the connection was attached. The response was dictated by two parameters, alpha and beta. Alpha controls the unloading stiffness, while beta control the reloading stiffness. The horizontal force in the connection at any given time was determined by multiplying the horizontal distortion by the current stiffness value. This response model was expected to more realistically portray the behavior of the flexible connection.

"Each panel was represented by truss members and was assumed to be rigid in plane, with attachment points located at the beam-column joints and at the midspan of the beams on two successive floors (because two panels were used from column-to-column, rather than a full-bay, full-story panel) Two distinct nodes having the same coordinates were specified at the upper flexible connection points. Two panels were assumed per bay to limit the complexity of the analysis. The first story was assumed to be open and no cladding panels were attached to framing members at the floor level."

The authors outlined the (DRAIN-2DX) program description and modifications, description of the study frame, and dynamic analysis. For the dynamic analysis, a triangular cyclic pulse was used as the input ground motion, with peaks at +/- 75 in/sec^2 occurring at 0.03 and 0.09 sec, and zero crossing points a 0.00, 0.06, and 0.12 sec. Using DRAIN-2D, the first, second, and third mode periods of the clad frames were found to be 0.40 sec., 0.13 sec., and 0.075 sec.

Analytical Results: "...top floor displacements for the frame model with cladding panels and connectors were found to be significantly less than those of the bare frame alone. [There are] also considerable displacement reductions at the lower floors due to added stiffness and damping provided by the cladding. Axial deformation of the beams is negligible, so displacement differences between the two frames will be similar along each column line.

"Also of interest are the ductility demands on the connections. All of the flexible connections experience inelastic behavior in the horizontal direction during the response period, with a maximum horizontal distortion of 0.03247 inches occurring in a connector on the second floor level at 0.0810 second. With the yielding distortion set at 0.0009 inches, the maximum translational ductility was computed as 36. Note, however that the connection yielding force was only 0.09 kips, which was later judged unrealistically low for a cladding panel connection. Increasing the yielding force to a representative flexible connection value would also decrease the maximum ductility. This will be explored more in follow on studies. With a bilinear response model, connection force levels are directly proportional to the connection distortions."

From Goodno and Craig [1990]:

Type of Study: Analytical.

Abstract: "Building cladding systems can play an important role in seismic response of modern highrise buildings provided that their connections are properly designed for earthquake forces. However exterior cladding is usually ignored in preparing a structural model for computer analysis because it is regarded as nonstructural. Recent research has pointed to the potential role that this precast concrete cladding can play, when properly designed, in providing lateral stiffness, ductility and energy dissipation to the overall building structure, especially during strong ground motions. Extensive modelling carried out by the authors and others have pointed to the critical role that cladding-structure connections play in the interaction between structure and cladding. Nonlinear time history analyses of both a two dimensional frame, in which the inelastic behavior of cladding connections was considered, and a three dimensional frame, which will eventually incorporate the connection model from the 2-D structure to represent nonlinear cladding response, are presented to demonstrate the potential benefits of cladding in reducing overall response. On-going experimental studies of promising cladding connections with beneficial stiffness and damping characteristics are also briefly described. Taken as a whole, these studies are intended to demonstrate the importance of including all major building components in computer analyses aimed at the realistic portrayal of overall building seismic response."

Conclusions and Future Research: "Two approaches for the analytical modelling of the dynamic hysteretic behavior of cladding connection components have been introduced. Both modelling approaches will be pursued in the future. The mechanical approach is favored by the authors based on the physical meaning behind it parameters and its ability to model the observed discontinuous behavior. But it is recognized that the simplicity of the three-parameter empirical approach makes it also very attractive. It is felt that the mechanical approach can be efficiently implemented only if coupled with an optimization technique for identifying the model parameters. Such a technique is being developed.

"Following the investigation of the cladding panel inserts, an advanced connector design will be tested and analyzed. The tests will isolate the behavior of the connector element from the influence of the insert and the building attachment elements. From these test results, nonlinear analytical models of the shear and flexural behavior of the connector and the insert-connector-attachment system will by developed using the approaches outlined in this paper.

"Once nonlinear hysteretic models have been defined for each of the three connection system components, the translational and rotational elements will be combined to simulate a complete connection system. Finally, the three components (insert-connector-attachment) will be tested together and the analytical assemblage will be calibrated against the results of the tests. The calibrated models of complete buildings being developed in other tasks in this research program

[Wolz, *et al.* 1992]. Analysis of these building models will determine potential cladding performance under strong ground motion conditions and will assess potential cladding influence on damping and on torsional and lateral response of buildings. Ultimately, the objective of this research is to formulate a methodology that will result in a new generation of cladding connection designs in which the cladding is an integral part of the lateral load resisting structural system of a building."

From Goodno, Craig, and Hsu [1991]

Type of Study: Experimental and Analytical.

Abstract: "The present paper describes the results of recent experimental and analytical studies of various cladding connection concepts used to attach precast cladding panels to a building structure. The work is part of a larger effort to examine how cladding interacts with a building structure during seismic loading, and to study how new or improved cladding designs may be used to provide additional response attenuation. The focus of the present paper is on the experimental studies and analytical modelling of enhanced cladding connections. The experimental work has involved the design and construction of a special test fixture that is capable of applying forces to a connection element that simulate seismic loading conditions. The measurements of connection performance are then used to develop several types of hysteretic models that are capable of representing the observed behavior. These models include purely empirical graphical formulations as well as generalized lumped parameter mechanical models. In the analytical work, the effect of selected advanced connections on overall building response is being studied using detailed structural models of several representative buildings. The present paper describes results from a 2D steel frame building model. Other models under study include a full 3D representation of a reinforced concrete building that experienced the Mexico 1985 earthquake."

Summary: "Studies to date have indicated that practical ductile cladding connections are achievable, and that their use in typical buildings could reduce response by as much as 70% for measured earthquakes. The results also strongly suggest that provision for ductile inelastic action in the connections for heavy concrete cladding panels can provide a level of seismic response attenuation comparable to those achieved by other means.

"The result of the research is expected to lead to simple and cost-effective solutions for advanced connections that will use standard construction materials fabricated in simple shapes that are easy to manufacture and reliable in service. Feedback and suggestions on the development of these designs from practicing engineers are essential to this process. The expectation is that cost-effective solutions developed along these lines will gain acceptance in the field."

From Pinelli, Craig, and Goodno [1991]: This paper appears to have much of the same content as Goodno, Craig, and Hsu [1991].

From Craig, Goodno, Pinelli, and Moor [1992]:

Type of Study: Experimental and Analytical.

Note: This paper is a continuation of the papers cited above. However, several new ideas for advanced connection elements are introduced, as seen in figure 4.5 (taken from fig. 2 in the paper). "Ductility and damping can be developed through a number of different passive processes including: Extrusion, inelastic connector action initiated through torsional or flexural element effects in the connection element, friction effects developed in slip processes for connectors designed with layered materials and fastened with bolts in oversized holes, and use of composite systems manufactured with material selected for strength and ductility... Keeping in mind that the ease of manufacturing and maintenance is as important as a good performance, a series of simple designs were formulated first. They all take advantage of plastification of the steel when stressed beyond yielding in flexure. Details are shown in figure 4.6 (taken from Fig. 3 in the paper). The connector consists of a section of square tube, 0.95 cm thick and 10 cm wide, cut away as shown to create two narrow flexural elements whose widths are tapered to initiate plastification over a greater portion of material."

The authors discussed the following: the test fixture, test programs (which included a hysteretic diagram in figure 4.7 (taken from fig. 6 in the paper) of the connector shown in figure 4.6; the connection model that includes a linear elastic element defined by stiffness, a bilinear element defined by its stiffness and yield load that provides the hysteretic behavior, and a 'gap' element defined by its stiffness and gap width that provides strain hardening; and the cladding system modelling.

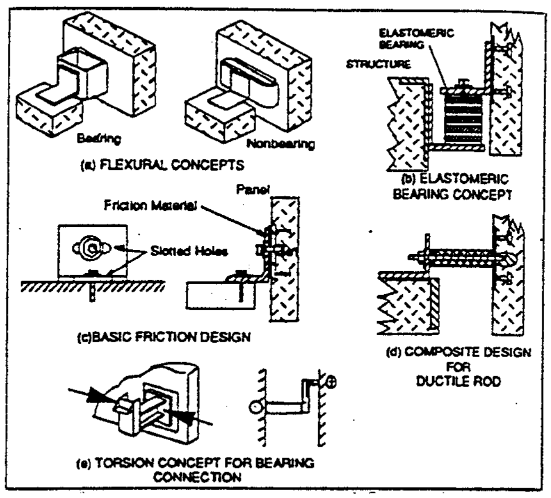


Figure 4.5. Examples of advanced connections (from Craig, Goodno, Pinelli, and Moor [1992]).

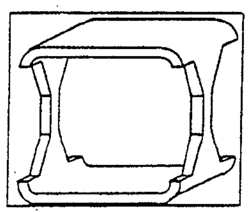
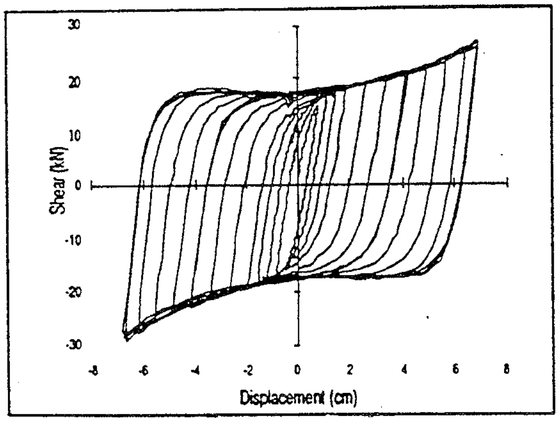


Figure 4.6. Hysteretic yielding connection (from Craig, Goodno, Pinelli, and Moor [1992]).

Figure 4.7. Hysteresis of specimen B (from Craig, Goodno, Pinelli, and Moor [1992]).



Conclusion and Future Research: "A machine has been presented that permits the testing of cladding connections under a reasonable approximation of their actual service conditions. A main advantage of the facility is its versatility for testing different kinds of damping connections, without geometric or attachment limitations and with or without inducing significant scaling. Another advantage is its ability to load the specimen in shear and flexure without inducing significant axial effects. The tests yield hysteresis plots from which the properties of damping, ductility, strength, and stiffness are evaluated.

"The results of the first tests carried out have been briefly presented. A series of additional tests, involving different types of energy dissipation mechanisms for the connector body in a connection system will follow..."

From Pinelli, Moor, Craig, and Goodno [1992]: This paper is a continuation of the papers cited above, and contains much of the same material as in Craig, Goodno, Pinelli, and Moor [1992], but with a bit more detail. The section on additional experimental observations follows:

Observations: The tapered connector as described in Craig, *et al.* [1992] was used. "...Several tubes of different thickness were tested [as noted in the previous reference]... Further tests showed that: (a) the specimen exhibits good fatigue behavior; (b) whether the specimen is bolted or welded has no influence on its behavior; and (c) the specimen can sustain gravity loads without losing its energy dissipation capabilities... In previous test reports on tapered specimens, the key issue has always been the provision of a sufficiently fixed condition for the ends of the tapered beams. The design presented here provides a simple and elegant solution through the reduced width of the flexural elements. In addition, the fact that the connector can be bolted to the insert with a single bolt (like many conventional connectors)makes it very attractive from an installation and maintenance point of view... Finally, the results indicate that some of these designs could be used for combined bearing and energy dissipating connections. Further results that assess the influence of physical dimensions, fatigue, and the fixation mechanism will be reported in a subsequent paper."

From Goodno, El-Gazairly, Hsu, and Craig [1992]:

Type of Study: Analytical.

Abstract: "...Analytical studies of two buildings [using DRAIN-2D], one a [2-D] 6-story steel building frame model used in laboratory studies of active and passive control systems, and the second an actual 12-story RC building damaged in the 1985 Mexico earthquake, were undertaken as part of a broader program of NSF-sponsored research aimed at understanding the role of non-linear interaction between cladding structural framing... The present paper presents the results of nonlinear dynamic analyses of the two case study structures which employ both conventional and advanced cladding systems for passive control of lateral response to earthquake ground motion."

Conclusions: "The analytical studies reported [in the paper], along with laboratory experimental studies described in a companion paper [Craig, *et al.* 1992, listed just above], show that heavy cladding systems which utilize ductile cladding connections can be used as an effective passive control system for buildings. These advanced cladding systems contributed to significant response reduction for the two case study structures considered in this investigation. The results suggest that provision for ductile inelastic action in the connections for heavy concrete cladding panels can provide levels of seismic response attenuation comparable to those achieved by other means. Cladding systems incorporating advanced connections hold considerable promise for passive control of building, perhaps as part of a hybrid active/passive building structural control system..."

From Pinelli, Craig, and Goodno [1992]:

Type of Study: Summary of work to date: "The paper describes the different phases of the development of advanced connections in both their experimental and analytical aspects. Test fixtures, program, and results are presented. Work to date has examined in detail the concrete insert component of a cladding connection and has resulted in the development of empirical and mechanical models. These models idealize the connection inserts as nonlinear rotation elements that incorporate the properties of stiffness degradation, pinching behavior, and related variations of the hysteresis loop area. The work is currently being extended to include the connection elements themselves, and as for the inserts the approach is to formulate both mechanical and empirical models on the basis of experimental measurements from laboratory tests. In all cases, agreement with the experimental results, model simplicity, and computational efficiency are the basis for the evaluation of the models which in turn will be used in continuing studies of full-scale building response."

From Pinelli, Craig, Goodno, and Hsu [1993a, 1993b]:

Type of Study: Experimental and Analytical.

Abstract: "Ductile cladding connections take advantage of the cladding-structure interaction during an earthquake to dissipate energy. An experimental test program studied the behavior of the different components of a connection system. Analytical models of the connection were incorporated into a 2D model of a six story building, both with and without cladding, to trace the response of the structure to earthquake excitations. Results show that properly designed energy dissipative connector elements can be responsible for the total hysteretic energy dissipated in the structural system. A design criterion for the connection that is formulated in terms of energy provides the optimal balance of stiffness and strength to be added to the structure by the dissipators. It results in maximum energy dissipation in the connectors, no plastification in the structural members, and reduced structural response. This approach could be applicable to both new and retrofitted buildings."

Introductory information: "It is only recently that precast concrete cladding has become a concern to engineers, due to numerous cladding failures, increased competition with other materials for facade enclosures, and renewed interest in methods for passive and active control to attenuate the dynamic response of buildings.

"During an earthquake, the behavior of the facade will be dictated by cyclic interaction between the panels and the supporting primary structure, and typically the connections are simultaneously subjected to three primary effects: (1) inertia forces generated by the acceleration of the panel; (2) interstory drift resisted by the panels which results in shear forces in the connection; and (3) gravity load of the panels which is supported by the bearing connections. While other forces may be developed, they are generally assumed to be of secondary importance...

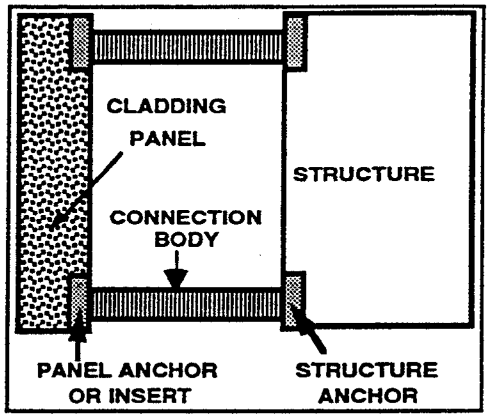
"This paper is concerned with a design criterion for such energy dissipative connections. An example of a case study building illustrates this criterion. An actual candidate for an advanced connection, thoroughly tested in the laboratory, is used as the case study. Although many practical problems remain to be solved, the design approach presented here demonstrates the potential for utilization of cladding as a structural element for both new building designs and retrofit of existing structures."

Cladding Connection System: "A cladding panel on a building facade is typically attached at four points, two at the bottom and two at the top. In U.S. practice, the bottom connections are usually bearing type connections while the top connections are usually tie-back connections. This arrangement is preferred by virtue of its simplicity, although it may lead to catastrophic failure in the case of failure of the upper tie-back connections... In this research, the U.S. model is considered.

"Although there are many different kinds of connection systems, all are generally composed of [see figure 4.8 taken from Fig. 1 in the paper]: (1) the anchor point, or insert, built into the precast panel, provides the panel anchorage; (2) the connection body (often a steel angle), or connector, forms the structural connection between the cladding panel and the main structure; and (3) the anchor into the building structure (a second insert or an attachment to a steel member).

"There is considerable variation in the design of each of the three major components depending upon the function of the connection (bearing or tie-back), the type of connection (welded or bolted), the architectural requirements, and other considerations [PCI 1988]."

Figure 4.8. Cladding system (from Pinelli, Craig, Goodno, and Hsu [1993a]).



Experimental Testing: The authors described their experimental testing program that has provided information on the behavior of cladding connections when subjected to combined shear and bending. They continued with an evaluation of advanced cladding connection designs. In figure 4.9a (taken from fig. 4a in the paper), they showed the details for one design. "The connector consists of a section of square tube, cut away as shown to create two narrow flexural elements whose widths are tapered to initiate plastification over a greater portion of material. The two tapered beams in flexure have a smaller maximum width through the cut-away than the fixed untapered elements to ensure that they will deform with double curvature. The connector could be placed between a panel and the supporting structure through a bolted attachment as shown in figure 4.9b [taken from fig. 4b in the paper].

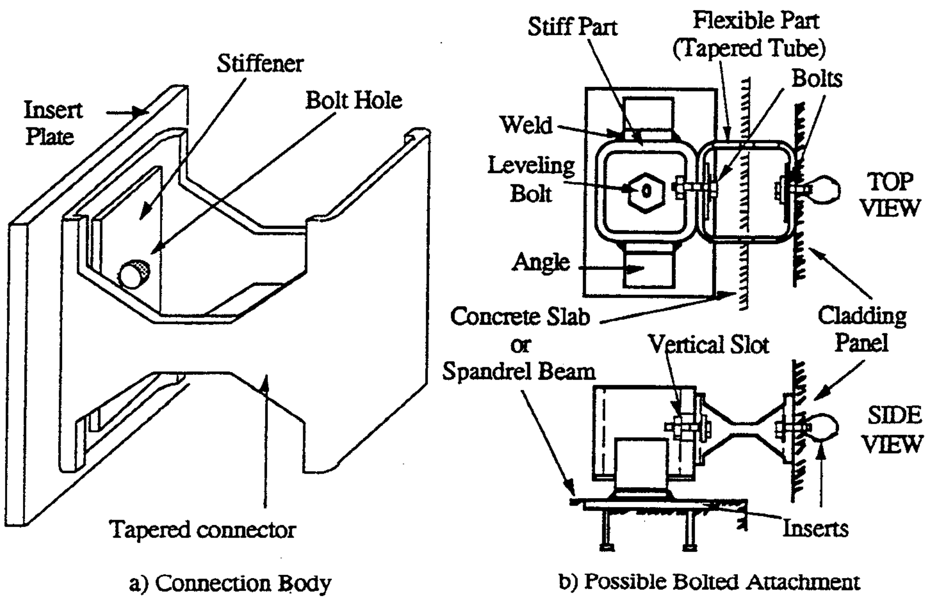


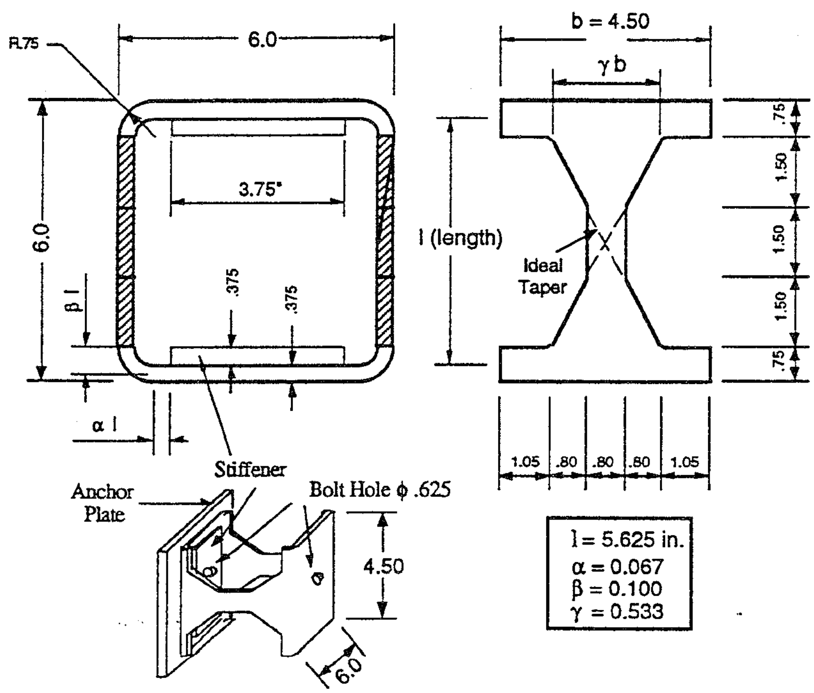
Figure 4.9. Advanced tapered connector (from Pinelli, Craig, Goodno, and Hsu [1993a]).

"Several tubes with different thicknesses were tested, and figure 4.10 [taken from fig. 5 in the paper] gives the geometric description of one of them. All the specimens were cut from 6 in x 6 in. (1520 mm x 152 mm) hot rolled square tubes, and the maximum height of the specimens varied from 4 in. (101.6 mm) to 4.5 in. (114.3 mm). In all the cases, the central portion of the tapered beams was cut straight to minimize the geometric discontinuity. The material was ASTM A500 grade B steel.

The testing of tapered connectors included quasi-static cycles of deformation. Tests were noted as "cycles of increasing amplitude to failure," "fatigue," "fatigue with increasing gravity load," and "cycles of increasing amplitude and fatigue." Tabulated test results include maximum displacement, maximum strain, and cycles to failure.

Six observations of the performance of the tapered advanced connections were made. In brief, they include: (1) advantageous hysteretic behavior; (2) good fatigue behavior; (3) no influence on behavior from bolting or welding; (4) sufficiently fixed condition for the ends of the tapered beams; (5) ductile failures; and (6) ability to sustain of gravity loads without losing is energy dissipation capabilities.

Figure 4.10. Geometry of 3/8" tapered tube TB 375 (from Pinelli, Craig, Goodno, and Hsu [1993a]).

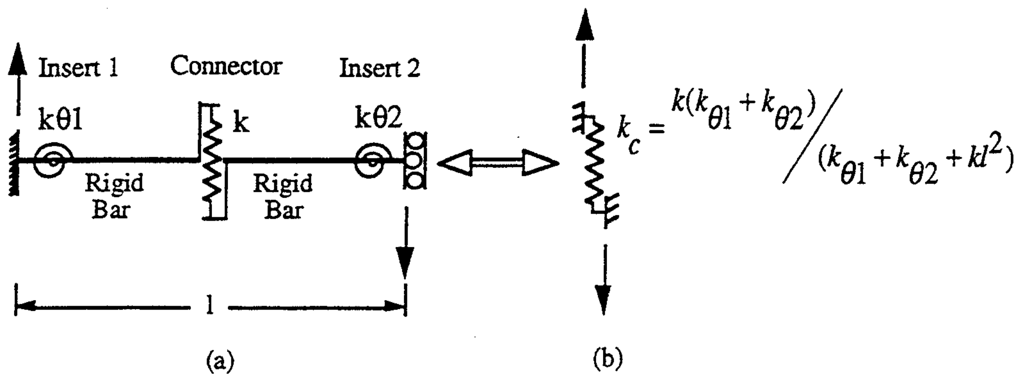


Analytical Modelling: "In order to test the validity of the proposed advanced cladding connection system, analytical models of the connection elements were developed and calibrated against the experimental results. These were then incorporated into a two dimensional structural model of a six story building that carries two heavy cladding panels per bay." (Note: The six-story building is shown with 3 bays in one direction, "scaled according to the artificial mass simulation method. No information was found in this paper or previous papers on the design criteria used to select the frame member sizes. The paper seems to indicate that the panels were included at the first story, too.)

"For the cladding-to-frame interaction studies, two cladding panels per bay were attached [to the analytical frame], with one fourth of their mass lumped at each panel node. The panels were assumed rigid and modelled with plane stress elements [using DRAIN-2D]...

"Each panel was attached to the structure by two rigid bearing connections at the bottom, and two energy dissipative connections at the top". The connection model is shown in figure 4.11 (taken from Fig. 10 in the paper). A further description of this model is given in the paper.

Figure 4.11. Schematic of the complete connection model (from Pinelli, Craig, Goodno, and Hsu [1993a]).



Design Criterion: "The study of the influence of variations in the stiffness and strength of a hysteretic connection on the response of a building suggests that a design would be best formulated in terms of energy dissipation. However, the dissipator also adds stiffness to the system, and therefore it changes the dynamic characteristics of the structure. In addition, the energy dissipation capacity of the dissipator is not a given fixed property. It is also a function of the excitation, which in turn depends on the modified dynamic characteristics of the structural system. The question asked is: What is the optimal balance of stiffness and strength to be added to the system by the dissipators that will result in a maximum energy dissipation, and a reduced response?

"An energy approach is based on resisting, or balancing the energy input to the structure by the excitation [earthquake] with the energy the structure is capable of absorbing. If the equation of motion is integrated with respect to the relative displacement from the time the ground motion excitation starts, the resulting 'relative' energy equation is: $E_i = E_k + E_s + E_d + E_h$, where E_i = the relative energy input tot he system by the earthquake; E_k = the relative kinetic energy; E_s = the recoverable elastic strain energy; E_d = the viscous damping energy; and E_h = the irrecoverable hysteretic energy term which corresponds to the energy dissipated through hysteresis during the motion.

"For a structure to resist an earthquake excitation, the sum of the energy terms on the right hand side of the equation must match the energy input on the left hand side of the equation. The sum of E_k and E_s constitutes the elastic vibrational energy. It will be uneconomical, if at all feasible, to rely exclusively on these energy two terms to balance the energy input. Instead, part of the energy can be dissipated through viscous or hysteretic damping. However, the hysteretic damping is often associated with yielding and damping to the structural members, formation of plastic hinges, and possible collapse of the structure. Alternatively, many of the so-called energy dissipators developed in recent years aim at concentrating the dissipation, either viscous or hysteretic, away from the structural members, in a few pre-engineered elements. This is exactly the idea behind the advanced cladding connections.

"In order to identify the best possible design for an advanced connection, the following criterion was adopted: the best connection will be the one that provides the higher ratio E_c/E_i, where E_c is the total viscous and hysteretic energy dissipated in all the connections on the facade, and E_i is the relative energy input to the structure at the end of the motion.

"At the same time, several constraints must also be satisfied [as follows]: (1) the ductility demand on any of the connections should not exceed an allowable value defined for each particular energy dissipator (e.g., from laboratory tests); (2) the connection should be able to satisfy the minimum code requirement regarding strength (e.g., UCB 1991, section 2337); and (3) the forces induced in the panel by the connections should not exceed the panel capacity.

"This criterion tries to take full advantage of the energy dissipation property of the connections. At the same time, energy is a variable that globally characterizes the damage potential of the earthquake for the entire structure, and does so more effectively than a displacement or interstory drift at a specific point. The remaining part of this paper shows that satisfaction of this design criterion will ensure that little hysteretic energy is dissipated in the structural members, and that the overall seismic response of the building is greatly reduced."

Validation of Design Criterion: "The validity of the proposed design criterion was tested by subjecting the building model to a variety of different ground motions. For each ground motion, the response of the reference case was investigated, and parametric studies were carried out: first for a hypothetical ideal elastoplastic connector (with a slight strain hardening to avoid numerical problems); and second, for the case of the actual tapered connector.

"Several different earthquake records were selected on the basis of differences in the frequency content, duration, and maximum acceleration. The intent was to investigate a case in which the fundamental frequency of the building would be below, very close to , and above the critical frequency range of the earthquake... In each case the seismic accelerations were scaled in such a way as to produce substantial damage to the reference case structure. The potential damage was measured by the amount of energy dissipated through plastification in the structural members, the magnitude of the top floor displacement, and the maximum interstory drift...

"The design criterion as stated above is, in fact, a constrained optimization problem. The objective function to be optimized (or maximized in this case) is the ratio of energies E_c/E_i, and the constraints are the several conditions listed before, and expressed as [follows]: (1) $c(1) = \mu_p$-50 if the maximum allowable ductility demand is chosen to be 50; (2) c(2) = 0.12-f_y where 0.12 kips is the scaled value of the minimum force requirement specified according to section 2337(b)4.B of the Uniform Building Code (UCB); and (3) c(3) = f_y-1.25 where 1.25 kips is the scaled upper bound placed on f_y to avoid damaging the cladding panels.

"It is assumed that all the constraints c(i) are satisfied as long as they remain negative. The decision variables are the stiffness, k, and the yield load, f_y, for each of the energy dissipative connectors. Thus, the number of decision variables varies with the number of different advanced connectors used on the facade. In nominally identical connectors are used for all the tie-back connections, there are only two decision variables (the present case)...

"For purely practical reasons, additional constraints must be added. Ideally.. tapered connectors could exhibit an combination of stiffness, k, and yield load, f_y, depending on their length, l, height, b, and thickness, t [see Figure 5 in the paper]. In reality, the tapered connector must also satisfy practical criteria related to constructability [as follows]: (1) the total length of the connector should be at least 6 in., for ease of manufacturing, and to accommodate typical spacing between the cladding panel and the structure; a shorter connector would not be recommended since a length decrease implies curvature and strain increases with negative consequences regarding fatigue; (2) the neck of the taper must ensure continuity between the two tapered parts of the beam; a value of 0.6 in. was adopted as the minimum acceptable value for a tie-back connections; and (3) the minimum thickness of the connector is defined by the minimum available thickness available for steel tubes; in this case this minimum was set to 0.025 in." For information on the solution of "this standard optimization problem," the interested reader is referred to the paper.

Conclusions: "...The design philosophy presented here for advanced cladding connections is based on an energy approach. The design seeks to maximize the dissipation of energy in the connections relative to the energy input to the structure. Such an approach results in an overall seismic response reduction along with preservation of the structural integrity of the framing members. The criterion required the use of software capable not only of carrying out nonlinear analysis, but also of computing the energy quantities involved in an earthquake excitation...

"The design method itself is a constrained optimization problem, where the constraints depend on code specifications, ductility requirements, the strength of the panels, and requirements specific to the type of dissipator being designed. The optimal design maximizes the energy dissipation in the connections. Different types of connections can be investigated with the same criterion. The adequacy of a class of connections can then be judged by how close it is able to approximate the ideal optimal design. Such an optimization problem can be implemented with relative ease with the help of existing optimization software.

"Ductility demand governs the selection of the optimal design. The analysis revealed that a tapered connector like the one presented in the paper might not be able to satisfy the ideal optimum conditions in every case. Nonetheless, an optimal feasible design can be defined in each case, and it will significantly improve the seismic behavior of the structure..."

Discussion of Pinelli, Craig, Goodno, and Hsu [1993a]: Cohen [1994a] wrote a discussion of Pinelli, *et al.* [1993a]. A few of the issues discussed in the paper and response are given here.

Cohen stated, "Cladding designers, even within the realm of a research project, must address at least the following issues: (1) architectural expression, (2) architectural technology, (3) architectural and structural aspects of cladding-to-frame detailing, (4) design and detailing of energy-dissipating cladding connections, (5) the relationship between the structural cladding and the structural frame and foundation, and (6) *building system* performance...

On architectural cladding design, Cohen noted, "...The behavioral differences among the [more common configuration of] one [versus] two panel per bay configurations with corner supports and panels with supports along the panel edges were not discussed by the authors."

On cladding-to-frame detailing, Cohen commented: "The most important aspects of architectural detailing include the elimination of (1) compression forces in the panels from column shortening, beam flexure, and differential thermal movement, (2) movement between panels which may result in differential "rocking" during a seismic event, and (3) out-of-plane panel movement. The panel-to-frame and panel-to-panel detailing is, in the words of several practitioners, 'terribly demanding and important.'"

Under energy-dissipating cladding connections, Cohen raised "Two fundamental and related structural design questions need to be raised: (1) How are the metallic yielding devices designed with respect to a given structural frame? and (2) Under what conditions do the metallic yielding devices yield?" She noted, "...The choices of connection stiffness and yield strength are a critical aspect of the design process. However, the authors do not present a transparent description on how this is accomplished. Is it recommended that practitioners understand and perform constrained optimization problems in order to design cladding connections, or is this only an intermediate step in research before a simplified design method is offered to practitioners?"

Response to Dicussion: Pinelli, *et al.* [1993b] addressed several of the comments made by Cohen [1993], some of which are noted here as follows: (1) The authors used their cladding pattern primarily to demonstrate a design methodology and not a specific application, citing tests previously done by Wang [1987]; (2) The authors stated that "the location of the attachment on the structure side can be moved with respect to the edge in order accommodate any desired gap (between the cladding panel and exterior surface of the structural frame)"; (3) The authors "suggest a way to minimize the compression forces through a vertical slot"; (4) The authors stated that "the advantage of the constrained optimization problem is that, if need be, additional constraints regarding displacements (interstory drifts) can be easily added to the problem"; (5) The authors mentioned that their "combination of elasto-plastic elements is not unduly complicated for the connection model..."; (6) The authors stated that "An optimal design reconciles [damping and stiffness] through the integration of nonlinear analysis software and optimization techniques..."; and (7) The authors said that there is a need to investigate the effect of using different connection properties at different stories.

From Pinelli, Craig, and Goodno [1994]: This paper is essentially the same as Pinelli, *et al.* [1993a], but with the following differences: For the possible attachments of the tapered advanced connector, the tapered part of the tube is explicitly labelled as the flexible part (see fig. 4.12 taken from fig. 3 in the paper). A new figure is given for the variation of hysteretic damping and ductility demand as a function of the decision variables, stiffness and yield stress (see fig. 4.13 taken from fig. 5 in the paper). "To solve the optimization problem defined by the design criterion, DRAIN-2D was combined with the program CONMIN [a Fortran program for constrained function minimization, developed by in 1973 by Vanderplaats at NASA's Ames Research Center]... Since no explicit functions exist for the objective function and the ductility constraints, their gradients were computed by finite differences, and the optimizer solved the optimization problem by the method of feasible directions."

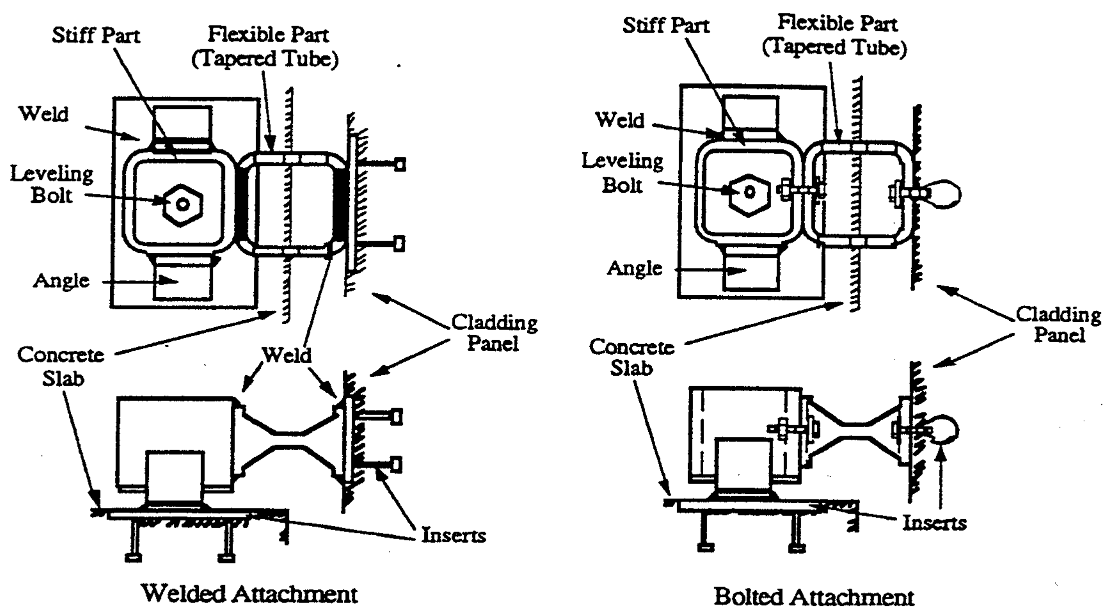


Figure 4.12. Possible attachments of advanced connector (from Pinelli, Craig, and Goodno [1994]).

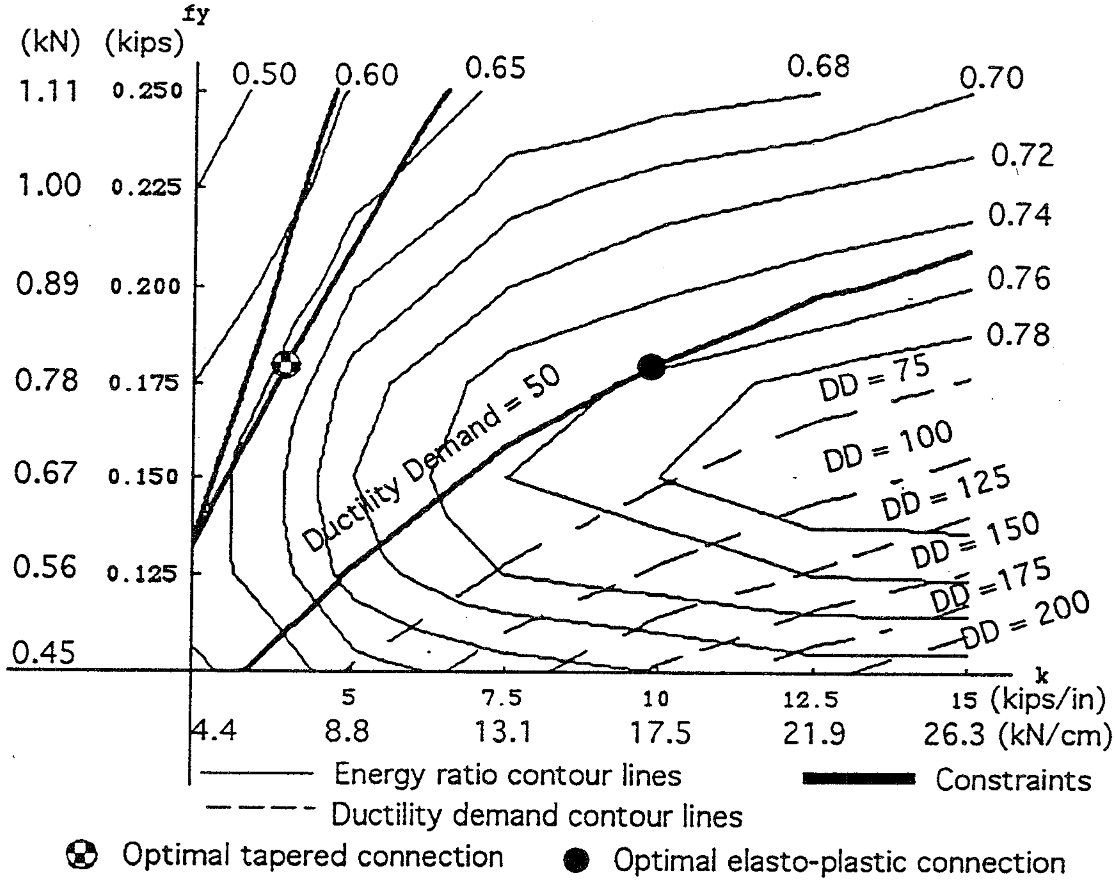


Figure 4.13. Variation of hysteretic damping and ductility demand as a function of k and fy (from Pinelli, Craig, and Goodno [1994]).

4.7d RESEARCH GROUP: School of Civil Engrg, Georgia Institute of Technology, Atlanta, Georgia

Topic of References: Hybrid control using cladding panels and connections.

References: • Goodno, Calise, Craig, and Sweriduk [1992]. "Hybrid Control of Building Seismic Response using Architectural Cladding." • Craig, Calise, Goodno, Hsu, and Sweriduk [1993]. "Active-Passive Damping for Structural Response Attenuation in Building Using Cladding." • Hsu, C.C.; Goodno, B.J.; and Craig, J.I. [1994]. "Hybrid Structural Control using Cladding Interaction with LQG Control."

Type of Study: Analytical.

Abstract: From Goodno, *et al.* [1992]: "Experimental and analytical investigations of hybrid control systems designed to combine passive damping provided by cladding-structure interaction with robust active control systems are being studied. The viewpoint in most of the studies to date has been to evaluate active control by way of comparison to passive control, rather than as a compliment to passive control. Hybrid control (combined active and passive systems) can be used to enhance the overall performance and reduce the individual problems associated with purely passive or active approaches. Experience gained earlier in the story of advanced cladding connection system designed to be employed in buildings as passive energy dissipation and stiffness augmentation systems is being combined with active control systems to realize a hybrid system with advantages over one in resisting earthquake loads. It is noteworthy that the control system design is being carried out using recent developments from modern robust control theory. The goal of the study is to develop a hybrid passive/active system that is capable of controlling the seismic response under the assumed worst-case conditions to be encountered. Initial work has been concerned with purely theoretical studies, but subsequent work will also include scale-model simulations in the laboratory... Results of the overall research effort are expected to contribute: (1) innovative approaches to robust control of building structures (both active and passive); and (2) innovative methods for combining engineered passive damping systems with modern robust controllers to achieve practical building control systems that can be applied to earthquake and other dynamic loading conditions."

From Craig, *et al.* [1993]: "Hybrid control systems combining passive damping provided by cladding-structure interaction with active control systems are being studied as a solution to the problem of seismic response attenuation in buildings. Previous studies by the authors have focused on using both mechanisms separately, but the objective of the present paper is to evaluate the effectiveness of a combined system. In this case, the control system, by itself, need not be designed to be powerful enough to completely control response, but instead it can allow permissible inelastic response. The paper describes how active control can be enhanced by passive control using optimally engineered ductile cladding connections. The resulting hybrid control system is shown to be more effective in controlling seismic response in a test building (NCEER 1/4 scale 6 story frame) than either active control or passive damping alone."

From Hsu, *et al.* [1994]: "The potential benefits and effectiveness of a hybrid control system for attenuation of response of building structures subjected to earthquake ground motion are investigated. The control system involved energy dissipating cladding connections and an active tendon system (ATS). If all the states of the structure are available, previous studies using a Linear Quadratic Regulator (LQR) design suggest that the hybrid system is more effective in controlling seismic response than either component system alone. In the paper, acceleration responses are assumed to be the only available measurements. The Linear Quadratic Gaussian (LQG) method is used and combined with Loop Transfer (LTR) to recover the desired loop transfer properties of the reference LQR design. The acceleration feedback control is implemented in the DRAIN-2D program used for nonlinear dynamic analyses of the passive system. Numerical results for the nominal system indicate that accelerations feedback is as effective as full state feedback and suggest that incorporation of an ATS and passive cladding damping in the structure can provide an innovative and practical approach for seismic response attenuation."

Summary and Conclusions: From Goodno, *et al.* [1992]: "The combined analytical and experimental study of hybrid active/passive building structural response control outlined herein is intended to: (a) develop promising passive damping mechanisms involving cladding-structure interaction; (b) combine this with active controllers capable of providing 'robust' performance; (c) study the performance and evaluate the relative effectiveness of several different hybrid systems that involved various distributions of passive damping and different control actuation systems; and (d) evaluate the practicality of these systems through experiments with small-scale laboratory models. The planned research will also include the design of more detailed experiments that will be proposed for future testing using larger scale (1/4 or greater) building models.

"Initial efforts in the passive damping area have resulted in the identification of candidate design for highly ductile and durable connections between precast cladding and the building structure.

"Preliminary investigations in the active control area have demonstrated that when using acceleration measurements alone, it is possible to achieve the same degree of performance afforded by full state feedback. Moreover, the robustness properties inherent in full state feedback can also be preserved by employing a loop transfer recover procedure that caused the loop transfer properties with acceleration measurements alone to approach that of a full state feedback design. In comparison to a full state design, the resulting output feedback design exhibits a nearly identical simulated earthquake response, and nearly identical gain and phase margins measured at the plant input.

"It is anticipated that the research will contribute to a more innovative approach to integrated structural design in which, by careful and deliberate intent, the cladding and an active control system combine to moderate the seismic response of a building. The research will contribute to a better understanding of the potential role which hybrid control systems can play in controlling the response of well-designed buildings in seismic regions. In this way, it is felt that more effective use can be made of all materials and building subsystems, and that the end results will be a more cost-effective and safe structure."

From Craig, *et al.* [1993]: "The preliminary investigation of a hybrid system incorporating a relatively simple LQR controller design with an Active Tendon System and passive cladding connection damping has been carried out. The results show very favorable performance for the hybrid system compared to either component system alone. This is evidenced through reductions in peak displacement response and required control forces. An analysis of energy time histories shows favorable reduction in energy dissipated passively in the connections when active control is present. These results suggest that a hybrid system might simultaneously reduce peak control force demand on the control system and maximum energy dissipation in the cladding connections compared to either configuration alone.

"These results are encouraging enough to suggest that more advanced robust controller design, starting with the LQG/LTR controller design developed in previous work [Goodno, *et al.* 1992] should be implemented in the fully nonlinear DRAIN-2D program. Moreover, the potential benefits provided by more robust controllers design using H_∞ and μ synthesis methods have yet to be explored.

From Hsu, *et al.* [1994]: "An LQG/LTR active tendon control mechanism is combined with previously developed engineered passive cladding damping to study the hybrid system performance under seismic conditions. The results indicate that: (1) the acceleration feedback is as effective as full state feedback; (2) the hybrid system has better disturbance rejection characteristics than either configuration alone; and (3) by deliberate intent, provides an innovative approach for seismic response attenuation.

"The LTR method is known to have an infinity norm greater than 1 [Calise, *et al.* 1993], so robust stability is not guaranteed for the specified set of uncertainties. It is further expected that in the presence of external disturbances, the structural parameter uncertainties will have a more pronounced effect on the control system design. This aspect will be addressed in more advanced robust controller design, H_∞ and μ synthesis methods, and full-order, nonlinear simulation. Also dynamic modelling problems and trade-off studies for active versus passive control are currently under investigation."

4.8 RESEARCH GROUP: Lorant Group, Phoenix, Arizona.

Reference: Kemeny, Z.A.; and Lorant, J. [1989]. "Energy Dissipating Elastomeric Connections."

Type of Study: Experimental.

Abstract: "Seismic forces on structures can be reduced by 1/4-2/3 by attaching architectural precast cladding (APC) to perimeter columns with connection isolators. These isolators provide 'sluggish moment connections' using field bolting only. They remain elastic (self-aligning) for moderate lateral loads and their steel parts yield when strong motion is induced. They decouple the APC mass until 1/2-3/4% story drift occurs, having 'wind-stiffness' only, but providing sufficient 'delayed stiffness and strength' for larger drifts and forces."

"These isolators provide joint flexibility with displacement limiter, ductility and redundancy in an economical and controllable manner. They can also accommodate temperature, creep, shrinkage and soil settlement movements."

"This paper introduces the principles and the proposed application of these isolators for APC supported by recent experimental results and a proposed design method based on current building codes."

The isolator and connection details are shown in figures 4.14-4.16 (taken from figs. 4-6 from the paper).

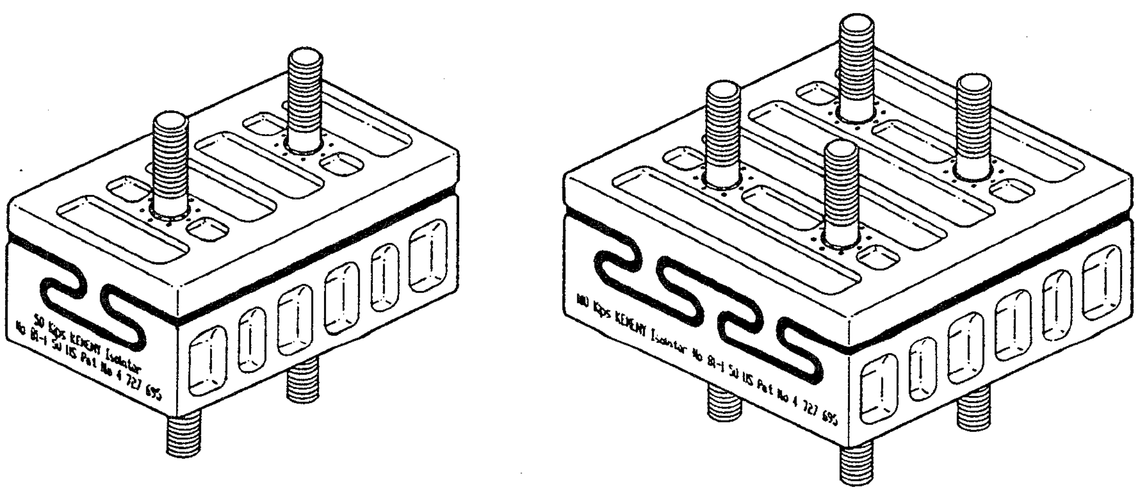
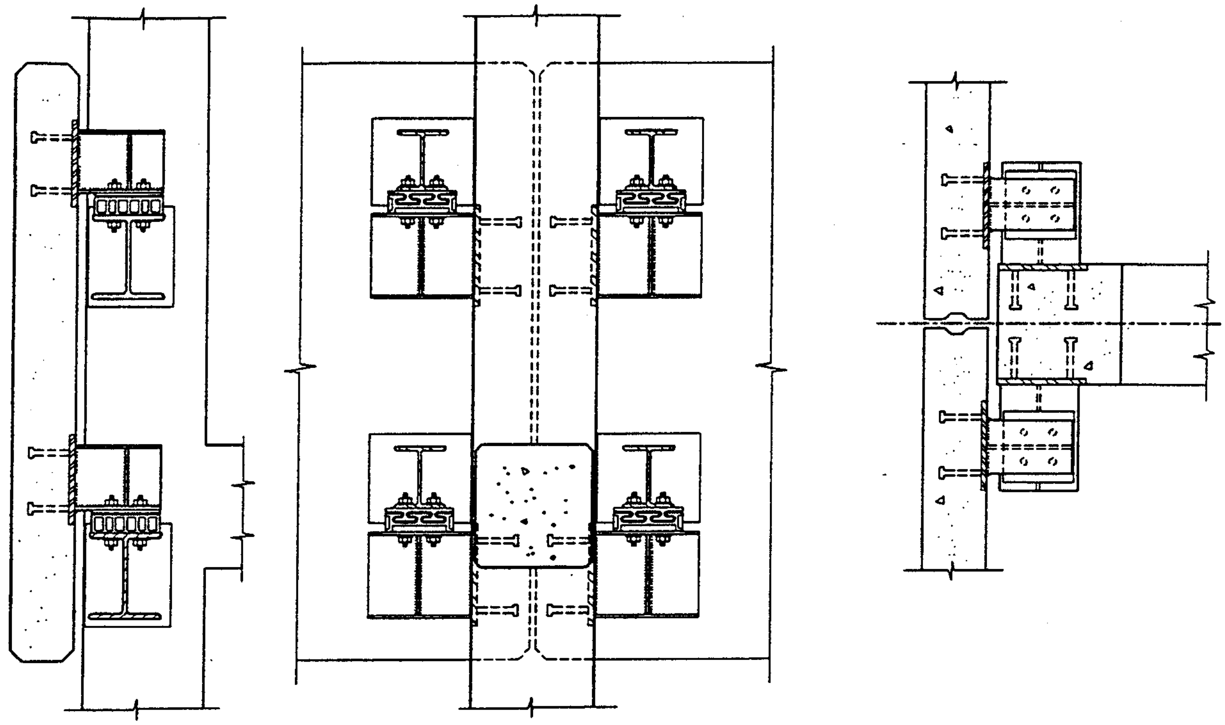
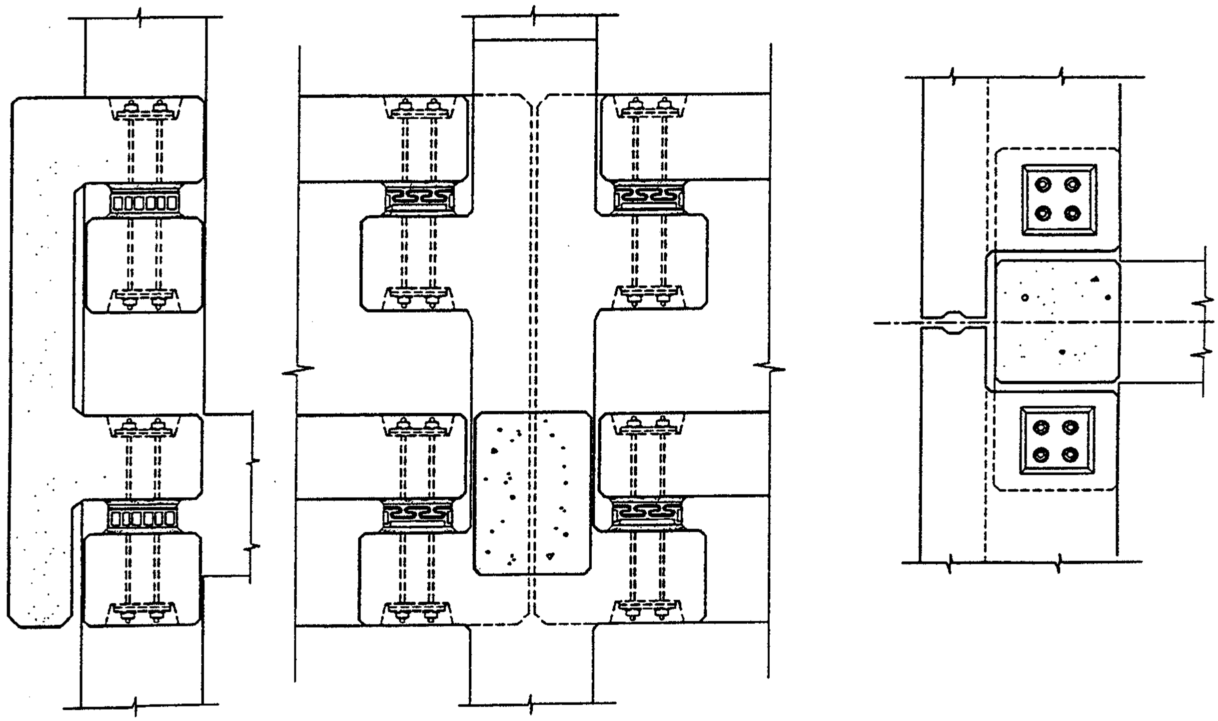


Figure 4.14. Interlocking keyed steel-rubber isolators (from Kemeny and Lorant [1989]).



a. panels through yard-welded short steel brackets



b. spandrels through conc. corbels

Figure 4.15. Structural details (concrete column) (from Kemeny and Lorant [1989]).

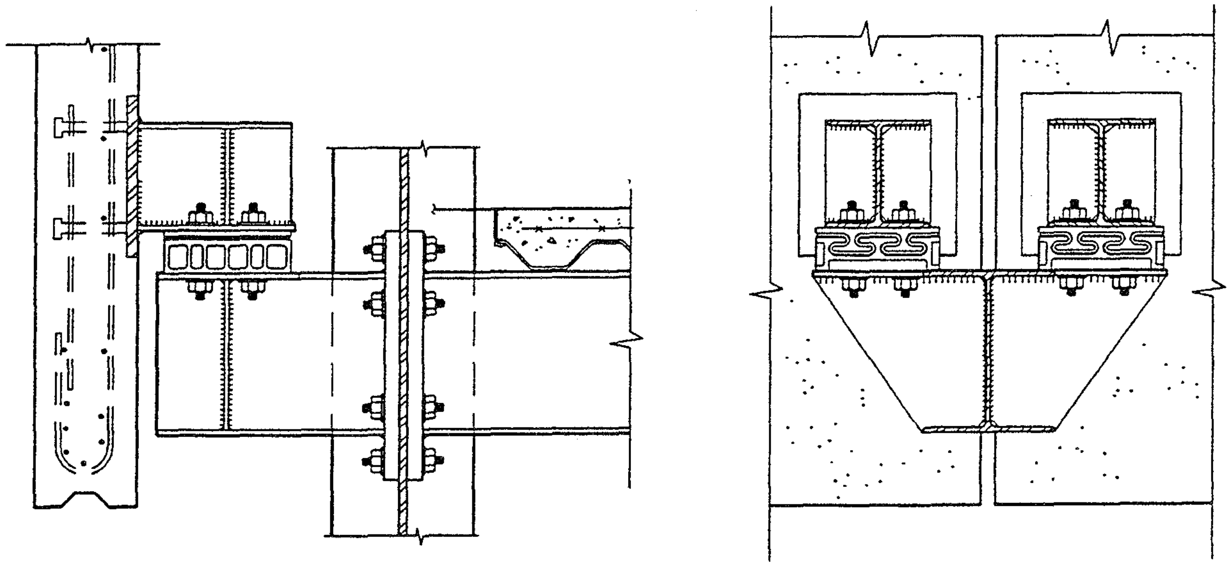


Figure 4.16. Structural details (steel column) (from Kemeny and Lorant [1989]).

Design Philosophy: "An innovative connection isolation uses energy dissipating (dampened) flexible connections (steel-rubber composites) designed for lateral load resistance systems in general, and as part of this for APC connections as well.

"Initially, when the elastomer is elastic and not yet compacted, the isolator has low stiffness, therefore, provides mass decoupling (isolation). However, when it becomes compacted between the steel teeth, these teeth continue to bend elastically and later yield, providing delayed strength and stiffness and finally, ductility by yielding."

The section on "design" is described. It entails the design of individual isolated connections by the use of several charts. The interested reader is referred to the paper for specific information.

Experimental Program: Single tooth isolators were tested.

Description of Test Specimens: The push-pull experimental set-up is shown in figure 4.17a (taken from Fig. 11a in the paper).

Type of Loading: Experimental hysteretic response curves were derived on the same single tooth isolate with increasing elastic stress and strain levels (4 Hz, sinusoidal, 66% max. rubber strain)... The rubber became compacted at 97% strain. The asymmetry is due to the different shape factors for uplifts (S = 8) and for bearing pressures (S = 4).

Main Findings: Experimental results on a single-tooth isolator are shown in Fig. 4.17b-4.17d and 4.18 (taken from fig. 11b-d and fig. 12 in the paper).

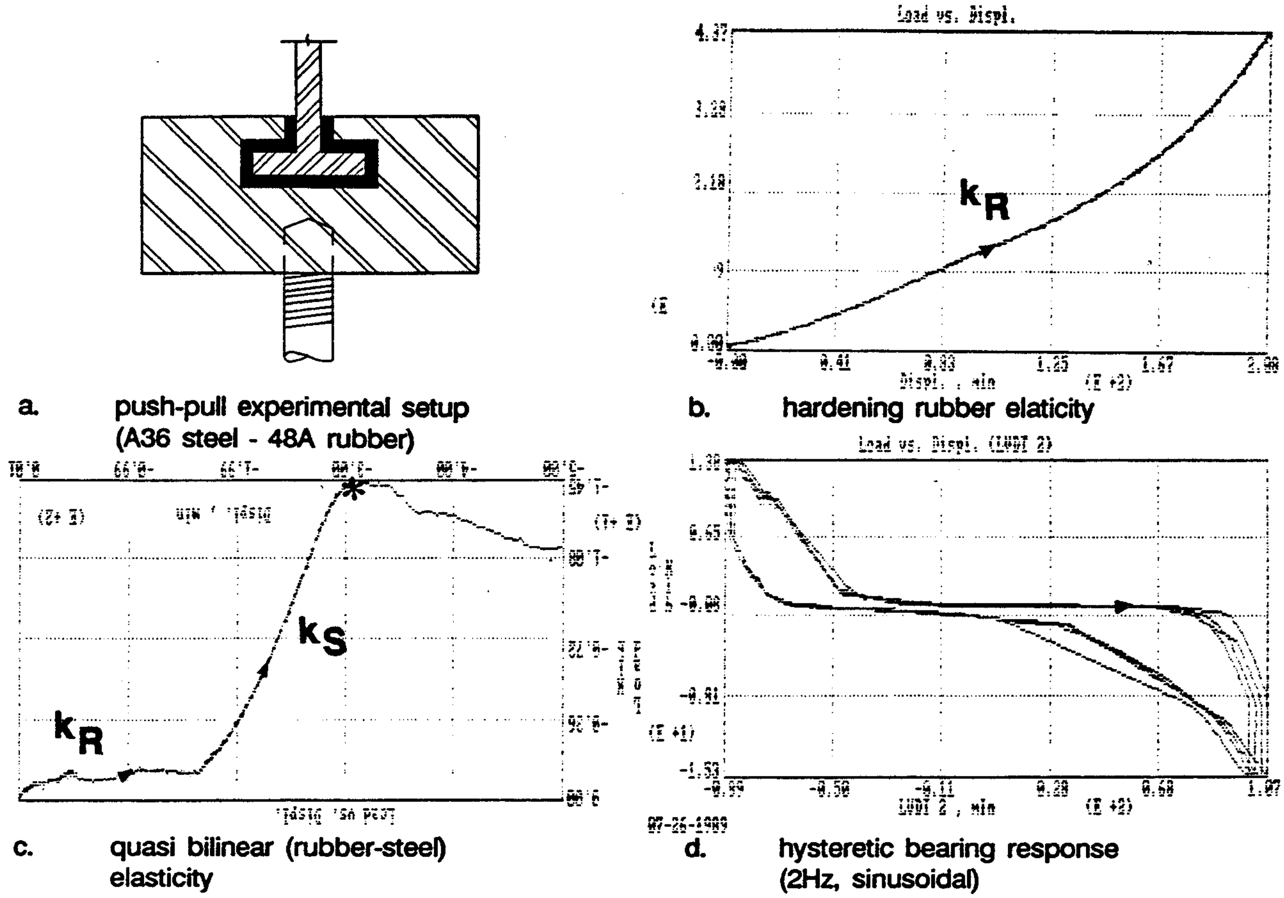


Figure 4.17. Experimental results on a single-tooth isolator (from Kemeny and Lorant [1989]).

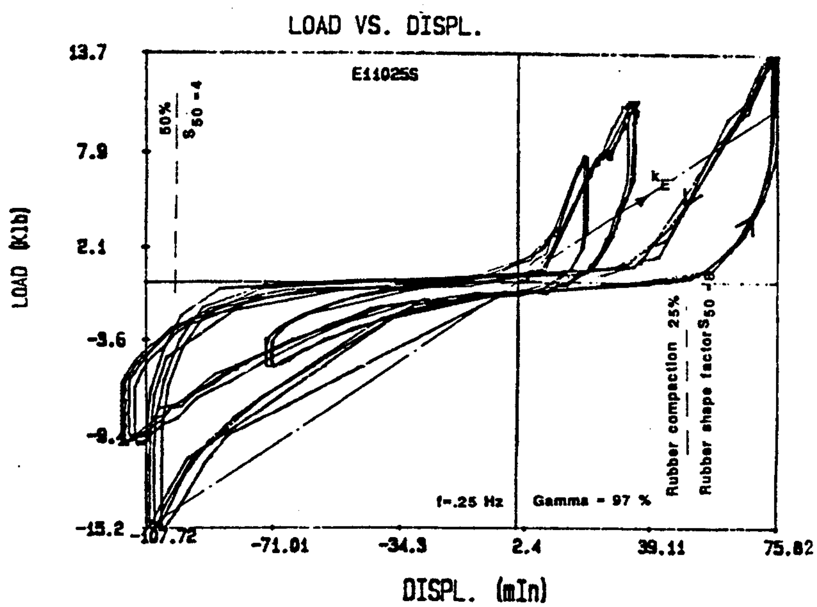


Figure 4.18. Experimental results on a single-tooth isolator (hysteretic) (from Kemeny and Lorant [1989]).

Conclusions: "Architectural Precast Cladding can be turned from purely dead load elements to lateral load resisting members of the structural system by appropriate isolated connections, like the one shown in this paper.

"The isolators do not reduce, but rather delay strength and stiffness. They might save 3 - 8% on seismic structures while considerably increasing seismic safety.

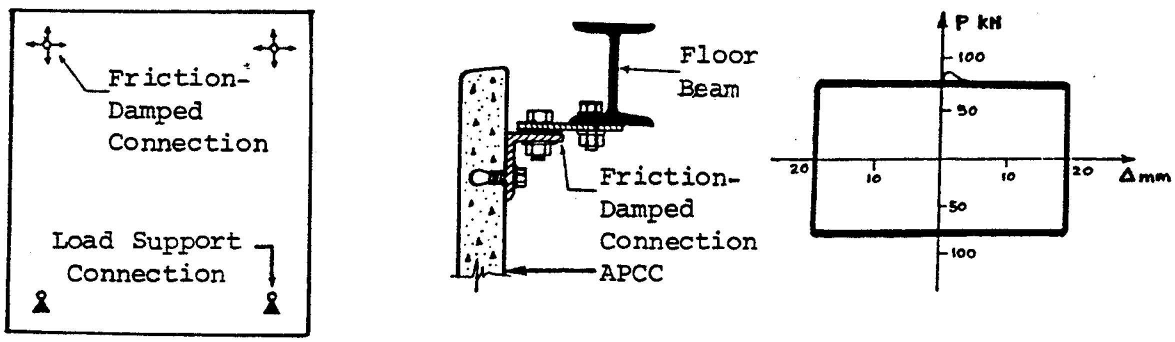
"Cracking is reduced in concrete and it is better able to accommodate temperature change and long term movements, although some additional cladding movement should be considered in detailing."

4.9 RESEARCH GROUP: Pall Dynamics Limited, Montreal, Quebec, Canada.

References: Pall, A.S. [1989]. "Friction-Damped Connections for Precast Concrete Cladding."

Type of Study: Analytical.

Abstract: "Inexpensive friction-damped connections have been developed to tie the architectural precast concrete cladding to the structural frame [see Figure 4.19 taken from fig. 1 in the paper]. The engineered connections ensure a controlled and reliable clad-frame interaction and introduce sufficient supplemental damping to safeguard against earthquake damage. Three dimensional non-linear time-history dynamic analysis has been used to demonstrate the superior seismic response of the friction-damped cladding system. During a major earthquake, a large portion of the seismic energy is dissipated mechanically in friction by the proposed connections with no dependence on the ductility, so the main structural elements remain elastic without damage. Since the rigidity and damping (are) distributed around the outer periphery of the structure, friction-damped clad-frames are highly resistant to torsion - a unique property unavailable in other structural systems. The proposed structural system, while assuring added safety to the occupants and reduced damage to the contents, offers the benefit of significant saving the initial cost of construction."



a). Location b). Typical Friction-Damped Connection c). Typical Hysteretic Loop

Figure 4.19. Friction-damped connection (from Pall [1989]).

Design Philosophy: "This paper describes an innovative friction bolted connection to tie the APCC to structural frame of concrete or steel. The engineered connections simulate an idealized elastic-plastic behavior. These patented connections are simple, reliable and need no maintenance or replacement over the life of the structure. These can be conveniently incorporated in the traditional connections. These connections slip under controlled conditions, relieve overstress due to volume changes by temperature or shrinkage and ensure required clad-frame interaction. The friction-damped connections are designed not to slip during service load conditions, wind storms or moderate earthquakes. During a major earthquake, before the elastic capacity of the frame or cladding is reached, the friction-damped connections slip at a predetermined load and dissipate excessive seismic energy during building motion. In this manner, the main structural members remain elastic without damage or at least yielding is delayed to be available during catastrophic conditions. As the claddings carry a constant load while slipping, the additional loads are carried by the moment-resisting frame. In this manner, redistribution of forces takes place between successive stories, forcing all the connections throughout the height to participate in the process of energy dissipation. In effect, the friction damped connections act as safety valves to limit the forces exerted on the frame of cladding and as structural dampers to limit the amplitude of vibrations."

Analytical Studies: To perform the analytical modelling, the slip load in the connections had to be chosen. The work done against friction in each story was determined from the consideration of maximum energy dissipation. The author computed the deflection of the moment-resisting frame as related to the portion of the horizontal shear carried by the frame. He then computed the maximum energy dissipation by differentiating the previously obtained equation, which "represents a condition in which, at extreme excitations, the shear force is shared equally by the frame and by the cladding connections. Thus the shear force causing the friction damped connections to slip is equal to the shear causing the frame to yield. However, it is desirable that the friction-damped connection should start slipping and dissipate energy prior to the frames or APCC reach yielding, say at 70% of their yield level." From this, the slip load in each connection is obtained. The response versus slip load relationship is shown in figure 4.20 (taken from fig. 2 in the paper).

Objectives: "Three dimensional non-linear time history dynamic analysis was carried out on a typical ten-story concrete frame office building of size 27m x 27m x 36 m high... to demonstrate the influence of the friction-damped connections on the seismic response of an APCC-frame."

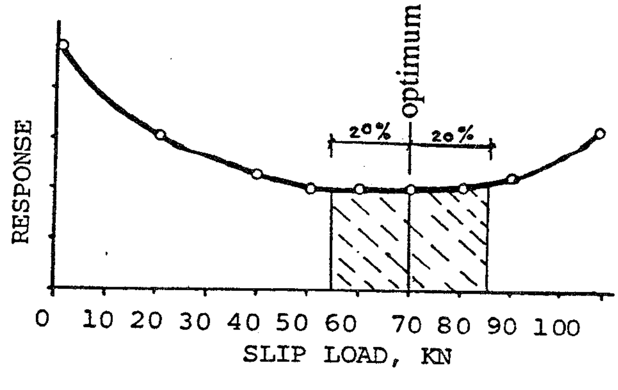


Figure 4.20. Response - slip load relationship (from Pall [1989]).

Assumptions: "The panels are connected at the top with 4 friction-damped connections. The bottom of the panel is connected with relatively stiff traditional load support connections."

Description of Analytical Models: "Viscous damping of 3% of critical was assumed in the initial elastic stage to account for the presence of non-structural elements. Hysteretic damping due to inelastic action of structural elements and slipping of friction-damped connections is automatically taken into account by the program. To account for a reduction in initial stiffness due to cracked sections, the gross sectional properties were reduced by 25% for columns and 50% for slabs. The Takeda model for degrading stiffness of flexural concrete members was used. Interaction between axial forces and moments for columns and P-Δ effect were taken into account by including geometric stiffness based on axial force under static loads. The integration time step was 0.005 second. A series of analyses were made to determine the slip load of the friction-damped connections to get the optimum response. The optimum slip load of each connection was 70 kN. However, there was a little variation in the response within ±20% of the optimum slip load."

Software: DRAIN-TABS

Ground Spectra and Ground Motions: "An artificial earthquake record, generated to match the design spectrum of Newmark-Blume-Kapur which represents an average of many earthquake records, was chosen as it does not favor any particular frequency."

Results of Analysis: "The effectiveness of friction-damped connections in improving the seismic response of the APCC-frame is seen in comparison to results with the unclad frame. The results of analysis are discussed below:

1. "Time-histories of deflection at the top of the building are shown in figure 4.21 (taken from fig. 5 in the paper). The peak amplitude for the unclad frame is 470 mm (0.0132H) with a permanent offset of 45 mm after the earthquake. At these story heights , it is expected that the architectural finishes and fixtures will be badly damaged. In the case of friction-damped APCC-frame, the peak amplitude is only 175 mm (0.0049H) and within acceptable limits. After the earthquake, the frame nearly returns to its original alignment.
2. "In case of friction-damped APCC-frame, column shears and column movements are only 60-70% of the unclad frame. All columns of friction-damped APCC-frame remained elastic without damage while 35% of columns are damaged in case of unclad frame.
3. "All slabs of friction-damped APCC-frame remained without damage with 45% of the slabs are damaged in case of unclad frame. In effect, friction, damped APCC-frame could have endured a higher intensity earthquake.
4. "Torsional resistance of friction-damped APCC-frame is 4 times superior to that of the unclad frame (see figures in paper). This is due to the fact that in the former system the rigidity and the the damping (are) distributed around the outer periphery of the building...
5. "Floor accelerations are reduced by more than 50%, hence damage to the secondary components, sensitive and expensive contents of the building can be avoided." (The response frequency of the clad frame is smaller than for the unclad frame. In addition, there appears to be significant higher mode response for the clad frame.)

6. "In order to quantify the performance of the friction-damped APCC-frame relative to unclad frame, viscous damping study was made. Stiffness and mass dependent damping was added to the unclad frame until the dynamic response of the unclad frame became equal to that of friction-damped APCC-frame. This equality was achieved by introducing 25% of damping. The percentage of supplemental damping increases as the intensity of earthquake increases."

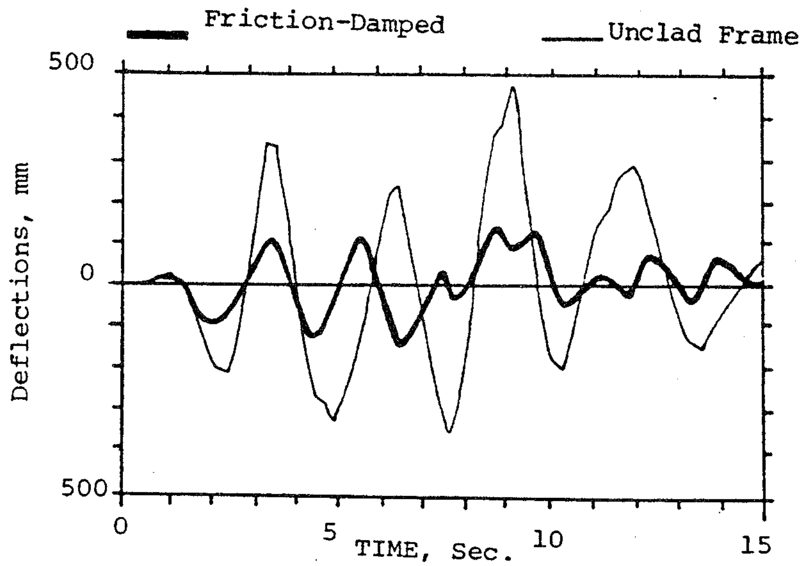


Figure 4.20. Time-histories of deflection at top (from Pall [1989]).

4.10 RESEARCH GROUP: Materials Department, Building Research Institute, Tsukuba-shi, Ibaraki, Japan.

Reference: Nishida and Ito [1988]. "Behavior of Nonstructural Elements Installed into Full Scale Steel Building"

Type of Study: Experimental and Analytical

Abstract: "This paper describes the results of aseismic tests of nonstructural elements [including precast (PC) curtain walls and glass fiber reinforced concrete (GFRC) panels]. Movement of rocking type curtain walls at static loading test agreed with the geometrical analysis."

Objectives: "The last phase on the full scale steel structure tests in the U.S.-Japan Cooperative Research Program aimed at confirming the seismic behavior of nonstructural elements... This paper describes the comparison of the seismic motion of nonstructural elements between the assumption in seismic design and the results of static loading tests."

Design Philosophy: Cladding panels and connection properties do not influence behavior of seismic-resistant framing. Cladding panels are designed for rocking or swaying type motion in-plane, by using bearing connections at the bottom panel corners and (vertical and horizontal) slotted holes permitting lateral movement at the top panel corners and panel rotation.

Experimental Program:

Description of Test Specimens: "The Japanese specimens were PC curtain walls, GFRC panels and other kinds of nonstructural elements, which represented the common construction practice in Japan. These elements were installed onto [the] 2nd-6th story of the moment-resisting steel frame. PC curtain walls were installed onto the 2nd-4th story on the frame [in the] loading direction. GFRC panels were installed opposite side of [the] PC curtain walls. Loading jacks applied one direction of horizontal displacements on the floor [from +/-$^{1}/1000$ to +/-$^{1}/40$] which resulted in the same story drift at each level. But the horizontal displacements of [the] 3rd and 6th floors were reduced to $1/\sqrt{2}$ (about 0.7) of other floor displacements.

Type of Loading: free and forced vibration tests; static loading tests.

Main Findings: "The test results on the motion of the PC panels agreed [fairly well] with the design assumption. The motion obtained from the test [is less than] the calculated value."

Analytical Studies:

Objectives: Estimation of geometrical panel movements.

Assumptions: Small displacement theory (no second order terms)

Description of Analytical Models: Approximate, to quantify the movement of the panels as derived form the geometric relation of the panel's rotation.

Software: None described.

Main Findings: Good comparisons were found between the test results and analysis for vertical movement of lateral connection on PC panel, and slip of vertical joint on PC panel.

Conclusions: "Concluding remarks are as follows: (1) from the viewpoint of [the] seismic design method of PC and GRFC panels adopting the rocking mechanism, the geometrical estimation could be considered to provide allowable story drift; [and] (2) in the case that the cladding is light, high modulus sealant could affect the free rocking motion of cladding."

CHAPTER 5

OTHER CLADDING MATERIALS FOR HEAVY PANELS

In this section, information and references are given on "heavy" cladding materials other than precast concrete with reinforcing bars or prestressed tendons. The "heavy" materials include glass fiber reinforced concrete (GFRC) panels, panels reinforced with new types of reinforcement, new types of reinforced concrete (RC) sandwich panels, and steel and steel alloy cladding panels. Excluded from this section are lightweight curtain walls.

5.1 Prefabricated Panel Systems

In addition to precast concrete cladding, Kuca [1993] noted that there are other pre-fabricated panel systems including: (1) laminated precast concrete; (2) panel systems; and (3) the steel strong back system.

"Laminated Precast Concrete: The use of thin stone for faces and the desire to find a method of erection that did not require labor intensive setting techniques on multi-storied buildings led to development of this system. This system provided the capability of providing a stone finish on a facade with the potential for time savings and inherent moisture resistance of precast. It combined a known system with the perceived quality of stone. The technology of engineering these two systems into a compatible unit involved the recognition of different coefficients of (thermal) expansion, anchorage provisions, bond breakers, and in some designs, the use of insulation integral with the panel.

"Panel Systems: These prefabricated systems of aluminum or steel in sandwich or siding provide the designer with an industrial design quality. Some systems have a highly refined appearance. These systems vary from the sophisticated moisture control concepts to the basic industrial panel. Custom panels are not readily available and therefore the designers flexibility is restricted to use of standardized components.

"Steel Strong Back: This system in the panel form like the Laminated Precast was developed in response to the need to provide a system that is erectable in larger elements for inherent economies. These panelized systems used steel members in various configurations with rolled shapes to metal studs to support stone, exterior insulation systems, tile, and brick. Another form of the system is the field erected steel or proprietary support system that is often used in limited applications for special areas."

5.2 GFRC Panels

A booklet entitled *GFRC: Recommended Practice for Glass Fiber Reinforced Concrete Panels* was prepared by the Precast/Prestressed Concrete Institute, PCI [1993]. The primary concern of PCI [1993] "is thin-walled architectural panels made of glass fiber reinforced concrete by the spray-up process under controlled factory conditions. These cladding panels are capable of accepting and transferring wind and self-weight and their own inertial seismic loads to the building's load-resisting systems, but are not considered as vertical loadbearing components or as part of the lateral load-resisting system."

According to PCI [1993], "Glass fiber reinforced concrete (GFRC) is the term applied to products manufactured using a cement/aggregate slurry reinforced throughout with alkali resistant glass fibers... It is important to understand that the material is a composite with reinforcing elements randomly distributed throughout the matrix, unlike reinforced concrete where the reinforcing steel is placed in tensile stress areas.

"GFRC cladding panels can be designed as wall units, window wall units. spandrels, mullions and column covers... GFRC architectural panels will generally weigh from 10 to 25 pounds per square foot (0.5 to 1.2 kPa) depending on surface finish, panel size and shape, and arrangement of steel stud framework... The low weight of GFRC panels decreases superimposed loads on the building's structural framing and foundation, usually providing savings in multistory construction and in areas with poor supporting soils... In building rehabilitation or retrofit projects, the use of GFRC panels for recladding minimizes the load added to the existing structure... Currently, GFRC is not considered as a vertical load-bearing component or as part of the lateral load-resisting system, although it can accept and transfer wind and self-weight and its own inertial loads to the building's load resisting system.

"Currently, the single skin GFRC panel is the predominantly used panel in the United States. These panels have a typical GFRC backing thickness between $^1/2$ and $^5/8$ in. (13 to 16 mm), not including the exposed aggregate for mix or veneer finish, when used. However, design requirements of panel size may call for a thicker backing or the use of stiffeners. In no case should the minimum design thickness of the backing be less than $^1/2$ in. (13 mm).

"Unless the panel has a functionally strengthening shape, GFRC properties dictate the use of stiffeners on panels of any appreciable size. Stiffeners commonly used include prefabricated, plant attached, cold formed steel studs or structural tubes; upstanding, single skin ribs formed on the back of the panel; and integral ribs formed on the back of the panel by spraying over hidden rib formers, such as expanded polystyrene strips. Each of these methods reinforce and stiffen the GFRC skin and provide a means for connecting the panel to the supporting structure... While each method of stiffening has advantages, use of steel studs in the most economical and preferred method for stiffening panels in the United States."

In PCI [1993], there is additional information on materials, including face mix and GFRC backing materials, reinforcement, stud frame and hardware, integral rib formers, welding, joint sealants and backer rods, and coatings. This is also information on physical properties, including factors affecting physical properties, tensile and flexural strengths, modulus of elasticity, compressive strength, impact resistance, shear strength, shrinkage and other moisture-induced movement, thermal movement, creep, freeze-thaw resistance, fire endurance, acoustical properties, density, thermal conductivity, permeability, and moisture absorption.

PCI [1993] contains background material on design philosophy, and panel stiffeners and methods of support. Several figures of methods of support (from PCI [1989]) are included here. Figure 5.1 (taken from fig. 26) shows a trussed rod gravity anchor. Figure 5.2 (from fig. 27) shows a plate gravity anchor. Figure 5.3 (from fig. 28) shows plate and trussed seismic anchors. Figure 5.4 (from fig. 30) shows spandrel connections. Figure 5.5 (from fig. 31) shows tie-back connections. Figure 5.6 (from fig. 32) shows a bearing connection combined with tie-back connection. Figure 5.7 (from fig. 34) shows story height panel concepts.

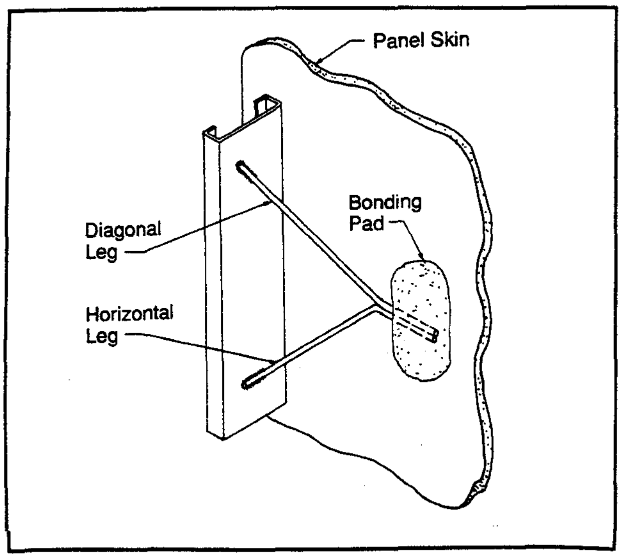


Figure 5.1. Trussed rod gravity anchor (from PCI [1993]).

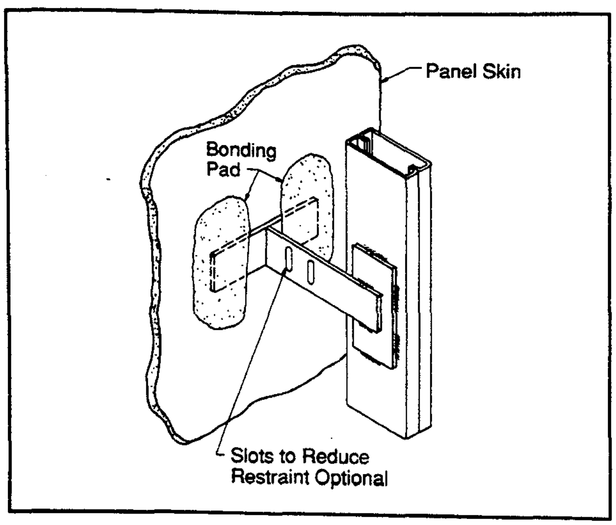
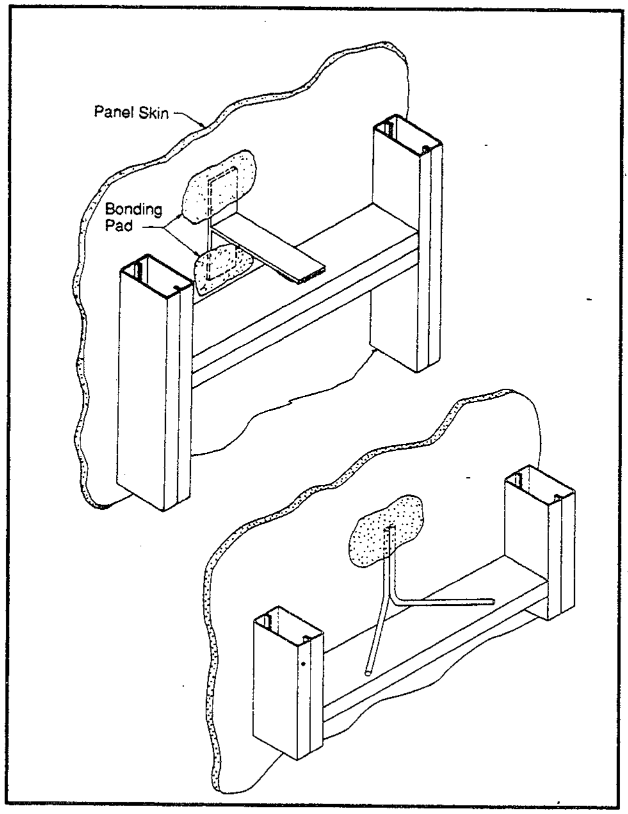


Figure 5.2. Plate gravity anchor (from PCI [1993]).

Figure 5.3. Plate and trussed seismic anchors (from PCI [1993]).



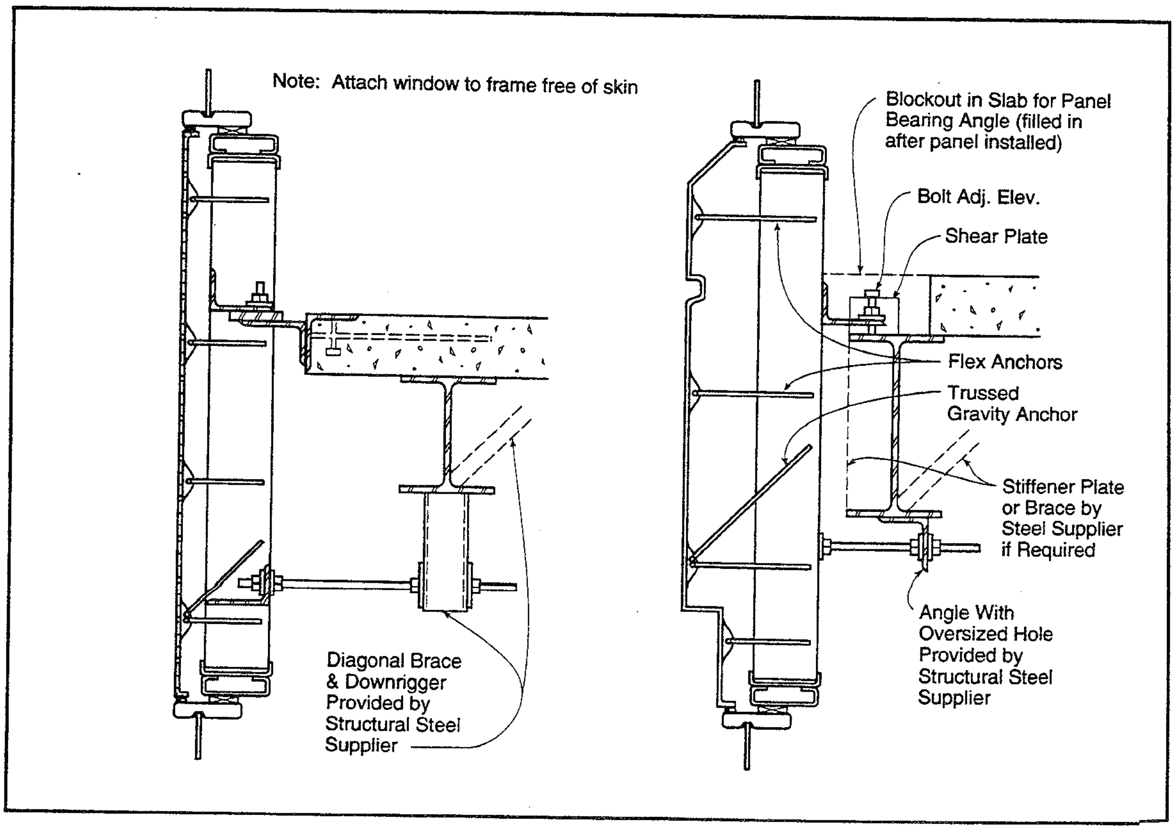


Figure 5.4. Spandrel connections (from PCI [1993]).

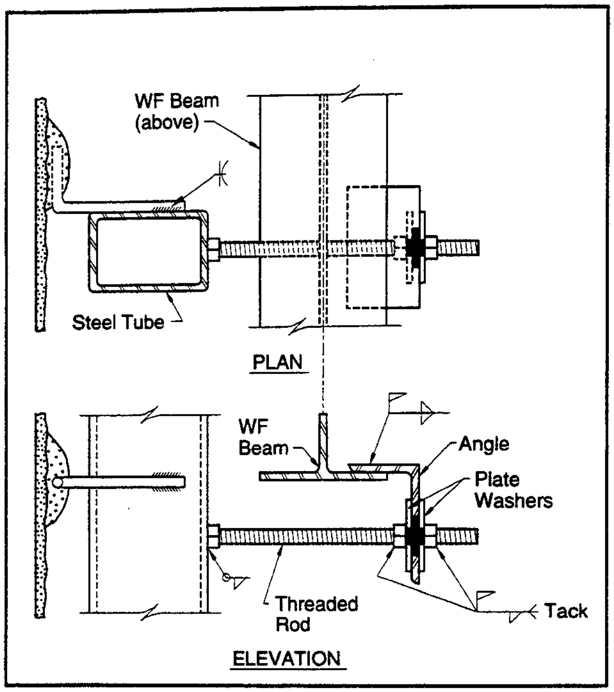


Figure 5.5. Tie-back connections.

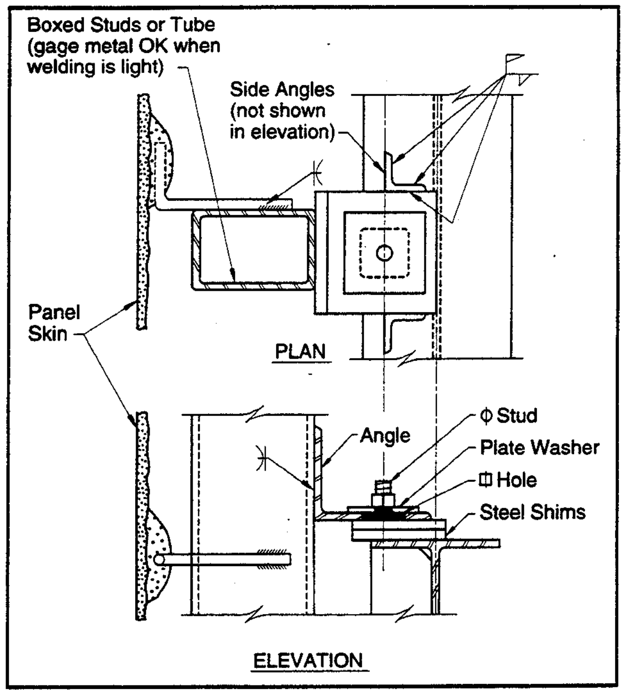


Figure 5.6. Bearing connection (combined with tie-back) (from PCI [1989}).

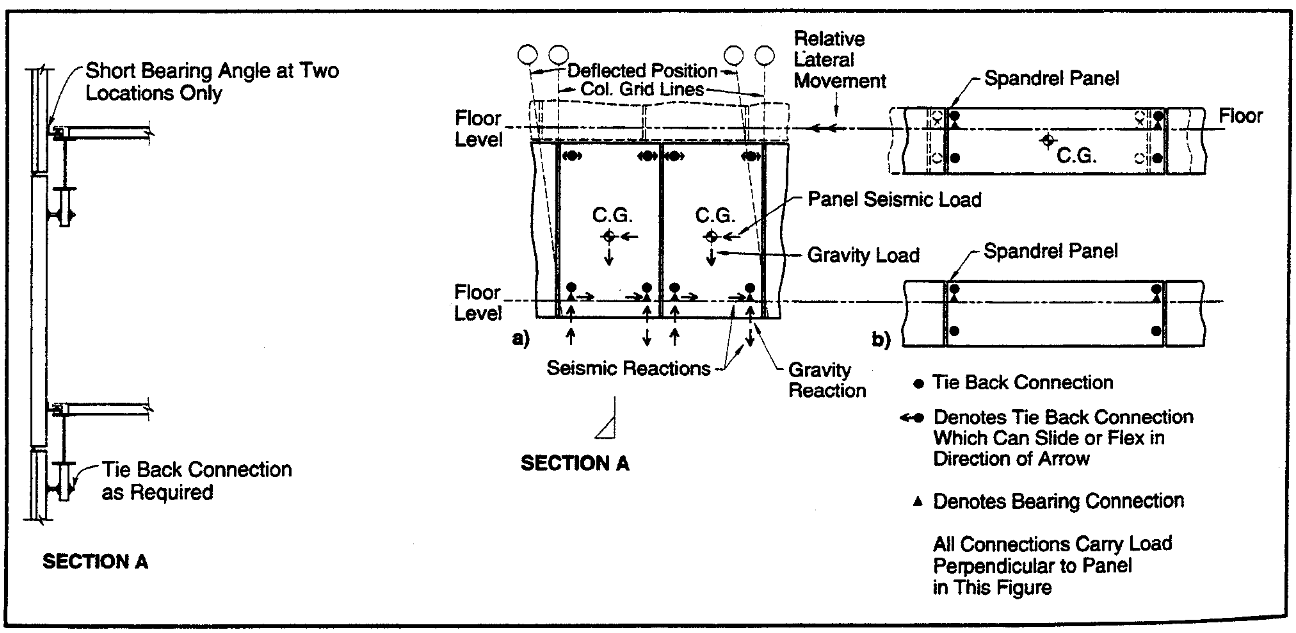


Figure 5.7. Story height panel connection concepts (from PCI[1989]).

In PCI [1993], "Design loads are also discussed, including panel service loads (which mentions earthquake load effects, noting "special considerations should be given to the three-dimensional panels where inertial forces can result in skin being stressed), and load factors and combinations. Limiting stresses (strength), including flexure and shear and tension are outlined. For shear and tension, it is noted that "Direct shear seldom controls the design of GFRC panels. Interlaminar shear, likewise, seldom controls the design of flat GFRC elements unless the span-depth ratio is less than 16. However, interlaminar shear may control design of connections. While in-plane shear occurring in the diaphragms and webs seldom controls design, it should be considered and the principal tension stresses limited..." There is also a section on deflection.

In addition, within the design information, there are written and graphic descriptions of panel types, stud frame system, inserts and embedment, etc.

In another reference, Harrison [1985], the design of anchor systems for glass reinforced *cement* cladding are outlined, including selection of metal, principles of anchor systems, anchor categories, factors governing design, movement of cladding and structures, corrosion, tightening of bolts, detailing and site realities.

5.3 New Types of Reinforcement

As noted by Gunnarsson and Hjalmasrron [1993], "The development of new high-tech materials for the construction sector has resulted in several interesting materials. Fiber materials such as glass fiber, carbon fiber, aramid fiber and other plastics were introduced in the concrete industry in the late 1970s. Their excellent properties in terms of strength, weight and durability in aggressive environments interested the engineers. At an early stage of development, short discrete fibers were successfully used by mixing uniformly with concrete in the manufacturing of precast concrete elements. In addition to short discrete fibers used in Fiber Reinforced Concrete (FRC) long fibers have also been developed, pultruded or braided, to replace conventional reinforcing steel. Out of these long fibers Fiber Reinforced Plastic (FRP) rods have been manufactured. These rods have been used in fabrication of precast concrete elements for facade or wall panels of buildings. Another promising field is prestressed concrete. Research has been carried out on many different applications. Several bridges and other utilizations employing FRP have already been built. Prestressed concrete structures will probably be the most important applications for RFP in the future construction industry."

The authors' report is comprised of two parts, Part 1 on aramid fiber reinforced plastics and miscellaneous applications, and Part 2 on static and impact flexural behavior of prestressed concrete beams reinforced with braided aramid fiber rods as PC-tendons. In Part 1, there are sections on the following: history, a brief description of fiber composite materials, material properties, aramid fiber reinforced plastic (AFRP), properties of the FiBRA-rod, properties of the Arapree strip, properties of the Technora rod, anchorage, application of fiber reinforced plastics, conclusions, and references.

These new types of reinforcement may become important when precast concrete cladding panels are designed to be engaged in shear, by locating the cladding-to-frame connections at more strategic locations to best take advantage of the shear strength of the reinforced concrete panel. These may also become important when detailing at the location of the inserts into the cladding panels become critical, that is, if the design criteria include a requirement that the concrete at/near the inserts exhibit minimal deterioration.

5.4 A New Type of RC Sandwich Panels

Einea, *et al.* [1994] have published a paper on "a new structurally and thermally efficient precast sandwich panel (PCSP) system."

"A new precast concrete panel system with a high thermal resistance and optimum structural performance has been developed. A hybrid truss provides the connector in this panel system - the diagonals are fiber-reinforced plastic bars and the chords are prestressed steel strands. Each connector (in the thru-thickness panel direction) consists of a fiber-reinforced bar fabricated in a deformed spiral shape through which a pair of prestressing strands is threaded to provide anchorage in concrete wythes (the wythes are two precast reinforced layers separated by a layer of insulation and joined with connectors that penetrate the insulation layer). The developed shear connecting system is described together with its advantages."

"An experimental and analytical investigation of the connecting system was conducted. The experimental program included testing of small scale specimens by push-off (pure shear) loading (in which each specimen is placed in a horizontal position and pushing the top wythe relative to the bottom one in a specially design steel frame), small scale specimens by flexural loading, and full scale panels by flexural loading. The analytical investigation included finite element modeling of the tested small scale specimens and comparison with theory of elasticity solutions. (A general finite element analysis program (ANSYS) was used with the appropriate two-dimensional library elements, with linear and nonlinear material models). Experimental and analytical results from finite element modeling and from theory of elasticity equations correlated well and showed that the developed panel system meets the objectives of the research and is expected to have a promising future."

Significant observations on shear and flexural testing are noted in the paper. For the analysis, "each component of the sandwich panel specimens is modeled using a general finite element analysis program (ANSYS) and the appropriate two-dimensional library elements," with linear and nonlinear material models. Observations and conclusions are outlined in the paper. One of the suggested topics for future research is "cyclic load testing to investigate the ductility and energy dissipation characteristics of the panels for use in high seismic risk areas."

5.5 Steel and Steel Alloy Cladding Panels

Cohen and Powell [1993] and Cohen [1994b,c,d] have conducted analytical studies, to date. The primary emphasis has been on the seismic retrofit of California seismic zone 4 buildings, with a secondary emphasis on new construction (due to mid-1990s economic conditions). The authors have taken a comprehensive, performance-based approach that entails consideration of 3-D systems-oriented thought processes involved in structural framing design. A comprehensive approach includes examination of soil-foundation-structure-nonstructure interaction. A performance-based approach has as its objective "damage control" to protect buildings and their occupants for anticipated levels of site-specific ground shaking. This approach is achieved by defining building-specific criteria or limits for at least two of the following levels: serviceability, functional or operational, and collapse or life-safety. An example of serviceability level might be defined by both a maximum allowable interstory drift ratio (to minimize nonstructural damage) and a maximum allowable horizontal floor acceleration (to maximize occupant comfort). An example of collapse or life-safety level might include strength criteria and a maximum allowable interstory drift limit of 2 percent. For an existing building, a possible criterion for consideration might be imposing both a limit on the rate of loading and an interstory drift limit (related to beam-end rotation) to protect potentially sensitive connections, such as welded steel connections.

Structural cladding and energy-dissipating cladding-to-frame-connections play an important role in seismically upgraded and new 3-D building systems (see Cohen [1995]). To be utilized as a reliable, safe sub-system of 3-D building systems, the structural properties of the cladding and connections must be incorporated into the overall design of 3-D building systems. The structural properties include strength, stiffness, connection damping, and connection cyclic deformation capacity including ductility. This means that the cladding sub-system needs to be studied as more than clad, fixed-based, 2-D perimeter frames, without recognition of the structural framing system and nonstructural features of existing and new buildings.

Before specific buildings were studied, Cohen and Powell [1993] critically examined current practice with regard to connection location and function. The proposed that connection functions should be distinct and the behavior should be simple. As a result, connection behavior would be more predictable and dependable. By locating the connections along the edges of the panels, rather than at the corners, there is a clear distinction between supporting the weight of the panels and transmitting shear forces from the panels to the frame. Also, if the demand is such that multiple metallic yielding devices or multiple viscous or viscoelastic devices are needed (depending on the performance criteria and deficiencies of the existing framing or the design intentions for new structural framing), several can be installed along the panel edges. In addition, the connections can be more easily detailed *not* to transmit compression forces to the panels.

Cohen and Powell [1993] presented details for the location and function of structural cladding-to-frame connections as follows:

(1) "Horizontal shears are transferred between the spandrel beams and panels through the connections along the horizontal bottom and top edges of the panels. At the bottom edges of the panels, the connections are designed to be elastic. At the top edges of the panels, the connections can be designed to be inelastic, and hence energy dissipating. (These connections remain elastic for wind loads and mild earthquakes.) Alternatively, the connections can be designed as viscous or viscoelastic (rate-dependent) dampers. The bottom and top connections are flexible in the vertical direction. This flexibility eliminates compression forces in the panels from column shortening, beam flexure, and differential thermal movement. The connections are assumed to have no rotational stiffness (in the analytical model).

(2) "Vertical shear is transferred between the columns and panels through the connections along the vertical edges of the panels. The connections are attached at mid-height of the columns. They also support the gravity load of the cladding. These connections are (vertically) short and fin-like, so that column shortening and differential thermal movement do not compress the panel. The connections are designed to remain elastic. They are flexible in the horizontal direction and have no rotational stiffness (analytically).

(3) "At each horizontal edge, separate connections are assumed for the panels in the stories above and below (i.e., an elastic connection for the panel above, and an inelastic connection or rate-dependent connection for the panel below). There is no direct panel-to-panel connection.

(4) "At each vertical edge, a single elastic 'fin' connection is assumed, connected to both the left panel and the right panel. This provides for force transfer from panel to column and also directly from panel to panel."

Additional information is available in Cohen [1994d] on the numbers of discrete locations of cladding connections along the top panel edge. In addition, there are comments on connection detailing, including the use of symmetrically placed connections to avoid torsion locally. For example, if viscous hydraulic dampers are used, then there should be at least two pairs, and within each pair, one should be extended when the other is compressed.

After defining connection location and function, Cohen and Powell [1993] and Cohen [1994b,c,d] developed a building-specific design procedure that could be easily transferred to practice, and then analytically studied the feasibility of using steel cladding panels and energy-dissipating cladding connections to improve seismic performance. (Typically, the panels are "heavy," ranging from 5/16" to 3/16" thick, with a stiffening grid at approximately 3 ft. on center with more significant stiffening around window openings and the edges of each full-bay, full-story panel. The area of glazing is on the order of 40% of the surface area of full-bay, full-story panels.)

To determine the role of structural cladding and energy-dissipating cladding connections, at least the following issues need to be addressed for seismic upgrading and new construction:

(1) Is the structural framing reinforced concrete or steel? What are some of the material-specific deficiencies?

(2) What are the strength, stiffness, and damping characteristics of the (existing or new) 3-D structural framing? What are the structural deficiencies globally and locally?

(3) Is supplemental damping needed within the interior of the building? Is the mass (and associated inertia forces) tributary to the perimeter framing, and the demands on the energy-dissipating cladding connections too large to be practical?

(4) What are the forces and deformations imposed on the perimeter foundation and framing?

(5) What is the role of the cladding in the redundancy of the framing system?

Cohen and Powell [1993] addressed the above issues and others for the design of a fictitious, new building that was designed with different percentages of code-required levels of lateral strength. These issues were addressed in Cohen [1994b] for a mid-1960s existing reinforced concrete building with a flat slab floor system and shallow beams only in the perimeter frames. Cohen [1994c,d] also addressed these issues as part of assessing what was needed to improve the multilevel performance of a late 1960s steel-framed building with a perimeter lateral load-resisting frame with built-up members and deep spandrel beams. In the latter two cases, before structural cladding and energy-dissipating connections can be considered, upgrading of the existing structures are required. In both cases, the cladding sub-system will prevent collapse, but cannot eliminate structural damage. For the particular steel building that was studied, column yielding cannot be precluded with the cladding sub-system, due to practical considerations of cladding design, detailing, and constructability. This will leave the building owner with a building that is safe for exit during/after strong ground shaking, but unusable due to a slight, but permanent, set manifested as a leaning building.

The interested reader is referred to Cohen and Powell [1993] and Cohen [1994b,c,d], in which design concepts are outlined for different roles of structural steel cladding and energy-dissipating cladding connections. Several analysis types are used to study the bare and clad frames. The most useful analysis type for these studies is nonlinear time history analysis, in which (force, deformation, and cyclic) demands on the connections, structural framing, and foundation) are examined using DRAIN-2DX, Version 1.10. Results from nonlinear time history analysis include envelope values and graphs of the following: interstory drift ratios; floor acceleration; force and deformation demands on structural framing members and connections, cladding panels, cladding-to-frame connections; dissipated work; etc.

Preliminary designs and details for the connections were developed from modifications to readily available energy-dissipating devices. Future work includes "working drawings" in which the connections are built and tested individually, and then installed behind panels in subassemblies with boundary conditions representative of those found in buildings. In addition, several new devices that are easily manufactured for use as energy-dissipating cladding connections will be considered.

CHAPTER 6

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